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EVALUATION OF THE INVERTED TEE SHALLOW BRIDGE SYSTEM FOR USE IN KANSAS

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16 Abstract

With the introduction of the pre-stressed concrete Inverted Tee (IT) girders as an alternative to the conventional concrete slab bridges, the distribution of live load in this system required considerable investigation. The approximate equations given in AASHTO LRFD can not be used for determining the distribution factors in the IT system because the required girder spacing conditions are not met. Therefore, there was a need for refined methods of analysis.

This report presents the comparison of the AASHTO LRFD and AASHTO Standard Specifications, ignoring the spacing conditions, with the results obtained from 2-dimensional grillage analysis and 3-dimensional finite element analysis. For this purpose, two software packages were used namely, RISA-3D for grillage analysis and GT STRUDL for finite element analysis.

The parameters that were included in this study were span length, superstructure width, skew angle, number of lanes loaded, end support conditions and overhang width. Based on this study, simple equations for determining girder distribution factors in IT bridges have been developed.

Additionally, the effect of using both the KDOT design procedures and AASHTO LRFD design procedures on the required number of strands was investigated.

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Final Report

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THE KANSAS DEPARTMENT OF TRANSPORTATION
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PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

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With the introduction of the pre-stressed concrete Inverted Tee (IT) girders as an alternative to the conventional concrete slab bridges, the distribution of live load in this system required considerable investigation. The approximate equations given in AASHTO LRFD can not be used for determining the distribution factors in the IT system because the required girder spacing conditions are not met. Therefore, there was a need for refined methods of analysis.

This report presents the comparison of the AASHTO LRFD and AASHTO Standard Specifications, ignoring the spacing conditions, with the results obtained from 2-dimensional grillage analysis and 3-dimensional finite element analysis. For this purpose, two software packages were used namely, RISA-3D for grillage analysis and GT STRUDL for finite element analysis.

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Chapter 1

Introduction

1.1 Background

The inverted tee (IT) bridge system is a precast composite concrete bridge superstructure system that was developed by Dr. Maher Tadros and other researchers at the University of Nebraska. This system uses pretensioned precast concrete members and has been shown to considerably reduce construction times and is structurally efficient for short spans. For replacing bridges that cross small streams or storm ditches, it is often desirable to increase the span lengths in order to minimize pier obstructions while maintaining the large span to depth ratio. This scenario has resulted in the use of cast-in-place (CIP) reinforced concrete bridges. The IT system is intended to provide an alternative to these CIP slab bridges. This system reduces the amount of formwork in the field and can be installed with relatively small construction equipment. However, the shallow depth of the IT's and the absence of a top flange for the base section could result in excessive deflections when larger spans are bridged with these members.

1.2 Scope of Research

The current AASHTO bridge codes [1] [2] address the distribution of live loads by providing equations for determining the fraction of load distributed to individual girders. However, neither of the codes address systems with adjacent composite precast girders like the inverted tee bridge system. It is very important to accurately estimate the load distribution factor for each individual girder. This report presents extensive computer modeling that was performed in order to evaluate the accuracy of current code equations (when applied to IT bridges) and to develop simplified distribution factors to be used with the IT system. The results of computer modeling are compared with the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications, 16th Edition and the AASHTO Load and Resistance Factor Design (LRFD) methods of computing distribution factors.

Both Skew and Non-skew bridges were studied. The research was also aimed at considering the type and position of barrier rails on the inverted tee bridges. The position of barrier rail was an important factor, as it affects the placement of the trucks and therefore the live load distribution factor. Also, preliminary design charts are developed and presented which illustrate the difference in design using AASHTO Standard Specification (16th Edition) and AASHTO LRFD (2nd Edition).

Chapter 2

Literature Review for Live Load Distribution Factor

2.1 Introduction

In a bridge superstructure, live load distribution factors are used to determine the fraction of a wheel load that is distributed to individual girders. This load distribution takes place through complex interactions between the girders and deck. Hence, different bridge codes have developed simplified methods to compute distribution factors. If these live load distribution factors are overestimated they may result in designs requiring larger members. Therefore, accurate distribution factor determination is critical for the new IT system. The AASHTO LRFD Specification addresses some of the variability in distribution factors for various bridge types by providing more comprehensive empirical methods and also by allowing the use of more refined methods of analyses. The following literature review will discuss the methods used for determining live load distribution factors and the previous research used to establish these design methods.

2.2 Literature Review

There are two mathematical idealizations that are frequently used for live load analysis of prestressed concrete and CIP deck superstructures. The first idealization, Grillage Analogy, consists of a discrete number of longitudinal and transverse beams. Longitudinal beam elements represent the prestressed concrete girders while the transverse beam members represent portions of the cast-in-place deck. In the second idealization, the Finite Element method, the girders are represented by discrete longitudinal members and the slab is modeled as a continuous transverse medium.

2.2.1 Tadros, Kamel and Hennessey

Tadros, Kamel and Hennessey developed the IT bridge system. They carried out research to determine the live load distribution factors for non-skew bridges using both the refined and the simplified methods. They found that the AASHTO Standard Specification values for moment distribution factors were close to the values obtained by refined methods, but the shear distribution factor obtained using AASHTO Standard Specifications were unconservative. There was less than 1 percent difference in the shear factor computed from grillage analogy and semi-continuum method. It was found that intermediate diaphragms have a negligible effect on live load distribution factors.

2.2.2 Zokaie, Osterkamp, and Imbsen

Zokaie, Osterkamp and Imbsen performed research on distribution of wheel loads on highway bridges, whose recommendations have been implemented in the AASHTO LRFD Specifications. The following formulae were developed for Beam and Slab bridge types.

Moment distribution to interior girders:

With multiple lanes loaded -

$$g_{\text{int}} = 0.15 + \left(\frac{S}{3'}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{Lt_s^{\,3}}\right)^{0.1} \qquad \text{(Equation 2.1)}$$

where,

g_{int} =distribution factor for interior girder for moment

S =spacing of beams, ft

L =span length of beam, ft

t_s =depth of concrete slab, in

K_g =longitudinal stiffness parameter =n(I+Aeg2)

where,

n =modular ratio between beam and deck materials

I =moment of inertia of beam, in4

A =cross sectional area of the beam, in2

eg =distance between centers of gravity of basic beam and deck, in

With one lane loaded-

$$g_{\text{int}} = 0.1 + \left(\frac{S}{4'}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{Lt_s^{\,3}}\right)^{0.1} \qquad \text{(Equation 2.2)}$$

Moment distribution to exterior girders:

With multiple lanes loaded -

$$g_{ext} = e \cdot g_{\text{int}} \qquad \text{(Equation 2.3)}$$

$$e = \frac{7' + d_e}{9.1'} \geq 1.0 \qquad \text{(Equation 2.4)}$$

where,

g_{ext} = distribution factor for exterior girder for moment

d_e =distance from edge of the lane to the center of the exterior web of the exterior girder, ft

With one lane loaded: It was recommended that simple beam distribution in transverse direction be used for single lane loading of edge girders.

Shear distribution to interior girders:

With multiple lanes loaded -

$$g_{\text{int}-V} = 0.4 + \left(\frac{S}{6'}\right) - \left(\frac{S}{25'}\right)^2 \qquad \text{(Equation 2.5)}$$

where,

$g_{int\text{-}V}$ = distribution factor for interior girder for shear

With one lane loaded -

$$g_{\text{int}-V} = 0.6 + \left(\frac{S}{15'}\right) \qquad \text{(Equation 2.6)}$$

Shear distribution to exterior girders:

With multiple lanes loaded -

$$g_{ext-V} = e \cdot g_{\text{int}} \qquad \text{(Equation 2.7)}$$

$$e = \frac{6' + d_e}{10'} \qquad \text{(Equation 2.8)}$$

where,

$g_{ext\text{-}V}$ = distribution factor for exterior girder for shear

With one lane loaded: It was recommended that simple beam distribution in transverse direction be used for single lane loading of edge girders.

Also, correction factors for calculation of interior moment and obtuse corner girder shear for skewed supports was suggested as follows-

Moment:

$$correction\ factor = 1 - c_1(\tan\theta)^{1.5} \qquad \text{(Equation 2.9)}$$

where,

θ =skew angle in degrees

$$c_1 = 0.25\left(\frac{K_g}{Lt_s^{\,3}}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5} \qquad \text{(Equation 2.10)}$$

If $\theta < 30^o$, $c_1 = 0.0$

If $\theta > 60^o$, use $\theta = 60^o$

Shear-

$$correction\ factor = 1 + c_1(\tan\theta) \qquad \text{(Equation 2.11)}$$

where,

$$c_1 = \frac{1}{5\left(\frac{K_g}{Lt_s^{\ 3}}\right)^{0.3}} \qquad \text{(Equation 2.12)}$$

The Range of applicability is as follows

$0^o \leq \theta \leq 60^o$

$3.5' \leq S \leq 16'$

$20' \leq L \leq 200'$

$4.5'' \leq t_s \leq 12''$

$10{,}000 \leq K_g \leq 7{,}000{,}000$ in^4

$N_b \langle 4$ (number of girders)

$-1' \leq d_e \leq 5.5'$

The final report of this research also suggested the positioning of the trucks to find maximum moments and shear values in a bridge. In addition, it also explored the different ways of generating computer models that can be used for refined method of analysis. It also suggests that plane grid analysis can produce sufficiently accurate results if modeled as per the recommendations.

2.2.3 Bishara, Liu and El-Ali

Bishara, Liu and El-Ali (1993) conducted research on developing expression for wheel load distribution on simply supported skew I-beam composite bridges for interior and exterior girders. Finite element analyses were carried out on bridges with different spans, widths and skew angles. The analysis took into account the 3-Dimensional interaction of all bridge members. They validated the results by testing an actual four lane skew bridge.

Wheel load distribution equations were developed for exterior and interior girders. These equations gave distribution factors, which were 20-80% of current AASHTO factor (S/5.5). Live load maximum bending moments in girders of skew bridges are generally lower than those in right bridges of same span and deck width. The maximum interior girder bending moment reduction increased with increase in skew angle. The distribution factor to the interior girders is practically insensitive to the change in length. The exterior girders become controlling in skew bridges as they are less affected by the skew angle effect, in case of bending moment. However, this tendency is only valid when the outer wheel of truck can be placed at 2 ft. from the centerline of the exterior girder.

Chapter 3

Methods for Determination of Live Load Distribution Factors

3.1 Introduction

On bridges, wheel load distribution takes place by interaction between the slab and the main longitudinal girders. The load is transferred from the deck slab to the longitudinal girders and then longitudinally to the substructure. Since slabs are typically continuous in the transverse direction (over the girders), the actual load path and therefore the amount of load sharing between girders cannot be readily determined. Therefore, bridge codes address this by providing empirical equations that give approximate values for transverse distribution of applied wheel loads. In the United States, such codes are developed by AASHTO. In 1993, the AASHTO subcommittee on bridges adopted a new set of specifications, known as the Load and Resistance Factor Design (LRFD). These new specifications changed both the loading magnitudes and the procedures used to distribute the vehicle loads. The equations given in AASHTO LRFD are believed to be more accurate for a broader range of bridges than the AASHTO Standard Specifications and have been checked using finite element analysis [4].

The LRFD Specifications allows the designer to use two different approaches in determining the Live load distribution factor. These two approaches are listed below-

(a) Use simplified approximate equations.

(b) Use refined methods like finite difference method, finite element method, grillage analogy, series or harmonic methods, etc.

The IT bridge system is unique since the girders are placed adjacent to each other and therefore do not meet the spacing criterion for the simplified equations in the codes. As a result a comprehensive analysis and comparison was performed using AASHTO Standard Specification approximate equations, AASHTO LRFD specifications approximate equations, and two refined methods of load distribution, namely, Grillage Analogy method and Finite Element Analysis.

3.2 AASHTO Standard Specification

3.2.1 Moment Distribution to Interior Beams and Stringers

The AASHTO Standard Specification allows for a simplified method of computing distribution factors. As per Table 3.23.1 (Distribution of Wheel Loads in Longitudinal Beams), for two or more lanes loaded case, the distribution factor can be calculated as S/5.5 (per wheel), where S is the beam spacing in feet. This equation applies to bridges with prestressed concrete girders supporting a concrete slab, with a centerline spacing of 14 feet or less.

3.2.2 Precast Concrete Beams Used in Multi-Beam Decks

Per section 3.23.4 of the AASHTO Standard Specification, more accurate distribution factors can be computed for precast concrete beams in multi-beam decks as actual section properties are used in computation. This section applies to a multi-beam bridge with prestressed concrete beams placed side by side (as done with the Inverted-T's). The conditions for this case to apply are that there has to be continuity developed between the beams through continuous longitudinal shear keys and transverse bolts and also that full depth, rigid diaphragms are provided at the ends. The fraction of wheel load that needs to be applied to obtain the live load bending moment is determined using the following equation

$$\text{Load Fraction} = \frac{S}{D} \qquad \text{(Equation 3.1)}$$

where,

S = width of precast member;

D = $(5.75 - 0.5N_L) + 0.7N_L(1 - 0.2C)^2$

when $C \leq 5$

D = $(5.75 - 0.5N_L)$ when $C > 5$

NL = number of traffic lanes

C = K(W/L)

where,

W =overall width of the bridge measured perpendicular to the longitudinal girders in feet;

L =span length measured parallel to longitudinal girders in feet; for girders with cast-in-place end diaphragms, use the length between end diaphragms;

K $= \{(1+\mu)I/J\}^{1/2}$

where,

I =moment of inertia;

J =Saint-Venant torsion constant

μ =Poisson's ratio for girders.

And

$$J = \sum \{(1/3)bt^3(1 - 0.63t/b)\}$$

Note, since there are no shear keys or transverse rods directly connecting the precast inverted-T's, this section technically does not apply.

3.3 AASHTO LRFD Specification

3.3.1 Simplified Method

The simplified equations in AASHTO LRFD Specification have been verified using finite element analyses and were found to be more accurate than the AASHTO Standard Specification equations for a broader range of bridge types [4]. The simplified equations for lateral load distribution of live loads are given in section 4.6.2.2.2 of the LRFD Specifications. The equations for live load distribution per lane for different conditions for concrete deck on concrete beams are as shown below. Note, these are valid only when the beam spacing, S, is between 1100 and 4900 mm.

Interior Girder Moment, two or more lanes loaded

$$g_{int} = 0.075 + \left(\frac{S}{2900}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{Lt_s^{3}}\right)^{0.1} \qquad \text{(Equation 3.2)}$$

where,

g_{int} =distribution factor for interior girder for moment

S =Spacing of beams, mm

L =Span length of beam, mm

t_s =depth of concrete slab, mm

K_g =longitudinal stiffness parameter $=n(I+Ae_g^2)$

where,

n =modular ratio between beam and deck materials

I =moment of inertia of beam, mm^4

A =cross sectional area of the beam, mm^2

e_g =distance between centers of gravity of basic beam and deck, mm

Interior Girder Moment, one lane loaded

$$g_{\text{int}} = 0.06 + \left(\frac{S}{4300}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{Lt_s^{3}}\right)^{0.1} \quad \text{(Equation 3.3)}$$

Exterior Girder Moment, two or more lanes loaded

$$g_{ext} = e \cdot g_{\text{int}} \quad \text{(Equation 3.4)}$$

$$e = 0.77 + \frac{d_e}{2800} \quad \text{(Equation 3.5)}$$

where,

g_{ext} = distribution factor for exterior girder for moment

e_g =distance between centers of gravity of basic beam and deck, mm

d_e =distance from edge of the lane to the center of the exterior web of the exterior girder, mm

Exterior Girder Moment, one lane loaded

Use Lever Rule

Interior Girder Shear, two or more lanes loaded

$$g_{\text{int}-V} = 0.2 + \left(\frac{S}{3600}\right) - \left(\frac{S}{10700}\right)^{2.0} \quad \text{(Equation 3.6)}$$

where,

g int-V= distribution factor for interior girder for shear

Interior Girder Shear, one lane loaded

$$g_{\text{int}-V} = 0.36 + \left(\frac{S}{7600}\right) \quad \text{(Equation 3.7)}$$

Exterior Girder Shear, two or more lanes loaded

$$g_{ext-V} = e \cdot g_{\text{int}} \quad \text{(Equation 3.8)}$$

$$e = 0.6 + \frac{d_e}{3600} \quad \text{(Equation 3.9)}$$

where,

$g_{ext\text{-}V}$ = distribution factor for exterior girder for shear

Exterior Girder Shear, one lane loaded

Use Lever Rule

Reduction of Load Distribution Factors for Moment in Longitudinal Beams on Skew Supports

$$correction\ factor = 1 - c_1(\tan\theta)^{1.5} \qquad \text{(Equation 3.10)}$$

where,

θ = skew angle in degrees

$$c_1 = 0.25\left(\frac{K_g}{Lt_s^{\ 3}}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5} \qquad \text{(Equation 3.11)}$$

If $\theta < 30^o$, $c_1 = 0.0$

If $\theta > 60^o$, use $\theta = 60^o$

Correction Factors for Load Distribution Factors for Support Shear of the Obtuse Corner

$$1 + 0.2\left(\frac{Lt_s^{\ 3}}{K_g}\right)^{0.3}\tan\theta \qquad \text{(Equation 3.12)}$$

There is a range of applicability for the above equations. This range is as follows:

$1100 \le S \le 4000$

$110 \le t_s \le 300$

$6000 \le L \le 73000$

$N_b \langle 4$ (number of girders)

It can be observed from the range of applicability that the minimum beam spacing is 1100 mm. This means that these equations are not meant to be used for beams adjacently placed.

3.3.2 Grillage Analogy Method

The grillage analogy method is one of the refined methods allowed by the LRFD to determine distribution factors for design. This two-dimensional method involves modeling the bridge superstructures as a planar grid of discrete longitudinal and transverse members. The number of transverse beam members needed is governed by the degree of accuracy required and by the position and type of loading applied. The longitudinal members are placed along the girder centerlines. In order to accurately model the bridge deck and supporting beams, proper connections are required between the longitudinal beams and transverse beams at the nodes, which were located at their intersection. Each of these nodes had three degrees of freedom; vertical translation perpendicular to the plane of the grid, and rotation about it's longitudinal and transverse axes. The boundary conditions at the girder ends were varied to determine the sensitivity of the model to the type of end restraints.

The moment of inertia of the longitudinal girders was assumed to be the composite inertia of the girder and with the contributing slab width, while the transverse girder inertia is taken as only that of the deck slab. The contributing slab width is taken as half the girder spacing on each side. Care was taken in determining the correct section properties. The Torsional stiffness of the prototype girders is the sum of the torsion of the parts that make up the girder. The torsional constant J, is taken as

$$J = \frac{A^4}{40.0 I_P} \qquad \text{(Equation 3.13)}$$

where,

A =cross sectional area of the composite beam, mm2

Ip = polar moment of inertia of the composite beam, Ip=Ix-x+Iy-y, mm4

3.3.3 Finite Element Analysis

Finite Element Method (FEM) is the other refined method (allowed by the AASHTO LRFD) that was used to obtain load distribution factors in this study. Although the 3-Dimensional finite element modeling provides a powerful method of analyzing simple to complex bridges, it was used primarily to verify the results obtained by the other 2D analyses.

The program selected (GT STRUDL) was capable of accurately modeling the bridge elements. The girders were formed from beam elements placed eccentrically below the deck slab that was formed from plate elements. The mesh density required depends on the desired accuracy of the results. Several densities were to be explored in order to determine the sensitivity of the model.

3.3.4 Rigid Body Effect for Exterior Girders

The AASHTO LRFD (Section 4.6.2.2.2d) states, "In a beam-slab bridge cross sections with diaphragms or cross-frames, the distribution factor for the exterior beam shall not be less than that which would be obtained by assuming that the cross-section deflects and rotates as a rigid cross-section". The recommended procedure for this is the same as the conventional approximation for loads on piles.

$$R = \frac{N_L}{N_b} + \frac{X_{ext}\sum^{N_L} e}{\sum^{N_b} x^2}$$ **(**Equation **3.14)**

where,

R =reaction on exterior beams in terms of lanes.

N_L =number of loaded lanes under consideration

N_b =number of beams

e =eccentricity of a design truck or a design lane load from the center of gravity of the pattern of girders, mm

x =horizontal distance from the center of gravity of the pattern of girders to each girder, mm

X_{ext} = horizontal distance from the center of gravity of the pattern of girders to the exterior girder, mm

Chapter 4

Development of Live Load Distribution Factors

4.1 Introduction

The investigation for the live load distribution factor was broadly classified into two categories, Non-Skew bridges and Skew bridges. Non-skew bridges were studied extensively for Moment and Shear distribution factors. In the case of Skew bridges, more emphasis was given to the determination of shear distribution factors, since these values increase for skew bridges. The software program used for the Grillage Analogy method was RISA-3D and for the Finite Element Analysis GT STRUDL was used.

4.2 Non-Skew Bridge

4.2.1 Discretization of Grillage Analogy Model

In Grillage analyses, the Non-skew beam and slab type of bridge is the easiest and most straight forward to model. The longitudinal members are placed along the girder centerlines, which represent the inverted tee girders, while the transverse members represent the stiffness of the slab. Typical discretization of the Non-skew bridge deck structure is shown in Figures 4.1 and 4.2.

Typically, the inverted tee beams have a spacing of 610 mm, center to center. Therefore, for both computation of the composite beam section properties and the transverse slab section properties, a slab width of 610 mm was also used. The exception to this rule was when the spacing of the beams is more than 610 mm. In this case, spacing between slab elements was kept at 610 mm.

4.2.1.1 Longitudinal Members

Typically, the bridges modeled were 6.7 m wide. Therefore, there were 12 inverted tees placed at 610 mm center to center. The section properties used to model the longitudinal members were the composite-beam section properties. Details of the composite beam are shown in Figure 4.3. All of the beams, both interior and exterior, had same section properties. The effect of the edge stiffening due to curbs was neglected. Table 4.1 gives the detailed composite section properties of the inverted tees evaluated in this research program.

4.2.1.2 Transverse Members

As noted above, the spacing of the transverse members was chosen to be 610 mm. The cross sectional properties were calculated for an un-cracked rectangular concrete section having a width of 610 mm and a height of 152 mm. Torsional constants were calculated using the equation introduced in chapter 3 {equation (3.13)}. Table 4.2 gives the cross-sectional properties of a typical transverse member.

4.2.2 Discretization of Finite Element Analysis Model

The finite element analysis more accurately represents a slab-on-girder bridge in the way it is modeled. In this analysis, the inverted tee girders are modeled as longitudinal members and deck slab as a continuous transverse medium. The transverse medium consists of number of plate elements of constant thickness. The desk slab and girder elements each had a different Young's moduli; for girder elements its was based on f'_c of 55 MPa and for slab on f'_c of 34 MPa and a Poisson ratio of 0.2

was used. Other inherent assumptions were that the materials were isotropic and the structural system followed linear elastic assumptions.

4.2.2.1 Longitudinal members

As previously noted, the bridges modeled were 6.7 m wide. In this case, all of the 12 longitudinal members, both interior and exterior, had the same section properties. The effects of concrete railings and the effect of edge stiffening due to curbs were neglected. The computer program GT STRUDL calculated the composite section properties from the sectional properties of the girders and deck thickness. The sectional properties of the girders are shown in Table 4.3.

4.2.2.2 Transverse Members

A standard 4-noded quadrilateral plate element of constant thickness of 152 mm was used in the modeling of the deck slab. An investigation was carried out to determine the effect, on accuracy, for changes in mesh size. Based on the investigation, a finite element mesh of 152 mm was selected to model for the slab. This model typically had more than 11,000 elements.

4.2.3 Different Bridge Models Analyzed

Different bridge models were created so that all objectives of the study were included. The different models that were created are described below.

(a) Simple span bridges with 610 mm girder spacing – This category represented the basic bridge type modeled. These bridges were 6.7 m wide (refer to figures 4.4 and 4.5 for typical models). The different combinations of simple span lengths and IT types that were modeled with

610 mm spacing are shown below.

Inverted tee girders used	Span (m)
IT 400	12.2
IT 500	18.3
IT 700	24.4
IT 900	30.5

(b) *Simple span bridges with varied girder spacing* – Bridge Models with IT 500 girders and 18.2 m spans were studied. The various girder spacing that were compared were 610 mm, 660 mm, 710 mm and 735 mm.

(c) *Simple span bridges with more than two design lane loads* – These bridges were 11 m wide with IT girders at 610 mm center-to-center spacing. They were compared with two loaded lanes case after applying the multiple presence factor (0.85) given in AASHTO LRFD section 3.6.1.1.2. Figure 4.6 shows a typical 11 meter wide bridge model.

(d) *Two span continuous bridges* – The same spans and IT girders used in model-type (a) were used for this investigation. In addition, a three span continuous bridges was modeled to verify the results obtained (refer to Figure 4.7 for a typical bridge model of this type).

4.2.4 Loading

The truck model used for the study was the AASHTO HS-25 as shown in figure 4.14. Determining the position for placement of the truck(s) (to create the maximum effect of moment o shear) was of prime importance since the objective was to determine the maximum girder response. The number of trucks placed on a bridge to produce this maximum response was of equal importance.

To determine the exact position of the truck(s) in the longitudinal direction, an analysis was performed on a single girder line with one truck wheel line. The trucks were then placed at the same longitudinal positions on the bridge model (where the maximum shear and moment values were obtained) in order to get the respective maximized responses on it. This position is near the support for Shear and near the mid-span for moment as shown in Figure 4.8 and 4.9 [4]. As shown in figure the maximized responses (shear and flexure) of a single beam line are determined for a single truck placed on it. The truck is then placed on the bridge model and the maximized girder responses (shear and flexure) are determined. The ratio of the girder response (shear or flexure) on a bridge model to that on the single beam line gives the value of the distribution factor. For example, the maximum girder response on a bridge model for shear is 108 KN and the maximum shear on a single beam line is 310.5 KN. Therefore, the ratio of 108 to 310.5 gives the shear distribution factor value as 0.348.

The transverse positions of the trucks also play an important role in determining the distribution factor. According to AASHTO LRFD, the first truck should be placed 610 mm from the edge of the design lane. The first truck was placed either on the exterior girder or the first interior girder. When it was placed on the exterior girder, it was assumed that there was an overhang of 610 mm. When it was placed on the first interior girder, it was assumed that the inside face of the barrier was at the center-line of the exterior girder. The first wheel line of the second truck was placed at a distance of 1.22 m from the second wheel line of the first truck [6]. Based on the values thus obtained a recommendation was made on the position of the barrier rail. Placing the trucks at 1.83 m from each other was also checked which yielded lower values of distribution factors. On the recommendation of the sponsors, the 1.22 m spacing was adhered to in the detailed investigation, since it produced conservative results.

4.3 Skew Bridges

4.3.1 Discretization of Grillage Analogy Model

The skew bridges were modeled according to the recommendations of the NCHRP report 12-26 [4]. The skewed decks complicate the manner in which the grillage mesh is laid out. A typical skew bridge model is shown in Figure 4.10. As recommended in the NCHRP report, spacing of transverse elements was adjusted so that the elements coincided with the girder ends (i.e. the support locations). Different support conditions were studied and the details of these supports will be given in a later section.

4.3.1.1 Longitudinal Members

The longitudinal members were placed coincidental with the girder lines as in case of Non-skew bridges. All the beams, both interior and exterior, had same sectional properties. The effect of the edge stiffening was neglected. The cross-sectional properties in the longitudinal direction were the same as those for the Non-skew Bridge given in Table 4.3.

4.3.1.2 Transverse Members

The members had to be laid out perpendicular to the longitudinal members (and not parallel to the supports) as recommended in the NCHRP report. The properties of the transverse members varied depending on the skew angle and position of the node where the longitudinal and transverse members intersected. Near the supports, the transverse members properties corresponded to the width of slab which was less than 610 mm for angles less than 45 degrees and greater for those above 45 degrees. In the middle portion of the bridge span the properties of the transverse members corresponded to that for a slab of 610 mm in width. To model the case where flexural cracking of the slab occurs, the transverse member properties were halved whereas the longitudinal properties were kept the same for simplicity.

4.3.2 Discretization of Finite Element Model

The skew bridge model is similarly modeled as the Non-skew model except for the fact that the transverse continuous medium, i.e., the slab is modeled parallel to the support. (See Figure 4.11)

4.3.3 Different Bridge Models Analyzed

The models that were created covered the complete range of skew bridges. The different skew bridges modeled had skew angles of 15, 30, 45 and 60 degrees. Different boundary conditions were applied which were as follows

(a) Standard case - No diaphragms, fixed for torsion, pinned for bending

(b) Diaphragm present – A diaphragm of width 914 mm and height 510 mm was used at the supports and also pinned for bending at the support.

(c) Pinned for bending and released for torsion, i.e., Moment Mxx released.

(d) Fixed for bending and Fixed for torsion, i.e., Moment Myy fixed

The effect of changing the transverse member properties to account for cracked slab section was given consideration wherein the section properties for transverse members were halved. Along with that the combined effect of cracked slab section properties and presence of diaphragm were investigated (designated as I_{eff}).

4.3.4 Loading

Unlike the non-skew bridges, the trucks were moved on the bridge model to determine the maximized girder response. The first truck was moved along the span and the positions for maximum girder responses were determined. Then, with this truck position fixed, the second truck was moved along the span (at a distance of 1.22 m from the first truck in the transverse direction) to find the maximum girder response for two trucks. This technique is explained by Barker and Puckett [6] and was done for both shear and moment.

4.4 Results and Observations

4.4.1 Non-Skew Bridges

(a) It can be seen from Table 4.4 that as the spacing is increased the value of distribution factor for both Shear and Moment increases.

(b) From Grillage analysis, the values of shear and moment distribution factors when the truck(s) is/are placed on the exterior girder are more than those obtained when placed on first interior (see Table 4.5).

(c) The values of distribution factors obtained using the AASHTO LRFD approximate equations are typically larger than those obtained using the refined methods for exterior girder loading and first interior girder loading (see Table 4.5).

(d) The AASHTO Standard Specification values obtained for Moment, when compared with refined methods, are conservative for the one lane loaded case with the wheel-line on the first interior girder. However, these values may be conservative for the shorter spans when two trucks are present as well as when there is an overhang (see Table 4.5 and 4.6).

(e) In case of continuous spans grillage analysis, it was observed that the positive and negative moment distribution factor values were approximately equal.

(f) From Table 4.7, it can be seen that the two lanes loaded case would be more critical than the three lanes loaded case on application of the multiplication factors suggested in the AASHTO LRFD specifications. A multiplication factor of 0.85 was applied to the values obtained from placing three trucks on the bridge model.

(g) The change in width of the bridge from 6.7 m to 11 m did not have any significance on the values of distribution factors when the same number of trucks are used (see Table 4.8).

4.4.2 Skew Bridges

(a) It was observed that the maximized response for shear would be obtained in the exterior girder obtuse corner.

(b) The position of the trucks to produce maximized effect is very critical. The NCHRP report suggests that in order to maximize shear response the trucks be placed close to each other near the supports. It was observed that the position of the trucks for maximized shear effect is when the trucks are positioned away from the supports for exterior girder, and one truck on the support and the other away from the support for the interior girder (see figures 4.12 and 4.13).

(c) The exterior girder gives greater value of shear distribution factor if the first wheel-line is placed on the exterior girder.

(d) The moment distribution factors obtained using the refined methods were always lower than the values obtained by using the AASHTO LRFD and AASHTO Standard Specifications equations (see Table 4.9). The first wheel-line was placed on the first interior girder when developing this table.

(e) The shear distribution factors obtained using the refined methods were greater than those obtained using the AASHTO Standard specifications. These same values were less than those obtained using the AASHTO LRFD equations for all interior girders, and also for exterior girders when the skew angle was less than 30 degrees. Distribution factors obtained by grillage analyses and finite element analyses were generally larger than those obtained by AASHTO LRFD for exterior girders when the skew angle was 30 degrees or larger (see tables 4.10).

(f) For zero skew angle presence of the end diaphragm did not make any appreciable change in the values obtained. However, for skewed bridges with skew angles less than 60 degrees, the presence of end diaphragms may greatly reduce the exterior girder shear near the obtuse corner.

(g) More realistic situations like presence of the end diaphragm and cracking of slab were also investigated. These typically gave lower values of shear distribution factors, then the standard condition of pinned for bending and fixed for torsion (see Tables 4.11 and 4.12).

(h) The values obtained using the two refined methods were usually within 10% of each other and often much closer.

(i) Shear variation was also studied on the bridge along the span. Maximized shear values were obtained in girders along the span of the bridge at the support, 0.1L, 0.2L, and 0.3L (where L is the span length) by moving the truck(s) along the span. It was found that the shear value is highest in the first one-tenth of the span. This is illustrated in Figures 4.15 and 4.16.

Table 4.1: Composite Beam Section Properties of the IT's studied

	Inverted Tee Beams			
	IT 400	**IT 500**	**IT700**	**IT 900**
Area, A(mm^2)	218710	234840	266450	298710
Moment of Inertia, I_{x-x}(mm^4)	$7.845x10^9$	$12.270x10^9$	$24.710x10^9$	$42.490x10^9$
Moment of Inertia, I_{y-y}(mm^4)	$5.508x10^9$	$5.542x10^9$	$5.610x10^9$	$5.679x10^9$
Torsional constant, J(mm^4)	$4.284x10^9$	$4.270x10^9$	$4.156x10^9$	$4.132x10^9$

Table 4.2: Cross-Sectional Properties of the Transverse Members

	Slab Member
Area, A (mm^2)	92903
Moment of Inertia, I_{x-x} (mm^4)	$0.180x10^9$
Moment of Inertia, I_{y-y} (mm^4)	$2.877x10^9$
Torsional constant, J (mm^4)	$0.609x10^9$

Table 4.3: Non-Composite Beam Section Properties of the IT's Studied

	Inverted Tee Beams			
	IT 400	**IT 500**	**IT700**	**IT 900**
Area, A (mm^2)	125806	141935	174193	205806
Moment of Inertia, I_{x-x} (mm^4)	$1.488x10^9$	$2.902x10^9$	$7.808x10^9$	$16.088x10^9$
Moment of Inertia, I_{y-y} (mm^4)	$2.617x10^9$	$2.652x10^9$	$2.719x10^9$	$2.788x10^9$
Torsional constant, J (mm^4)	$0.687x10^{99}$	$0.824x10^9$	$1.098x10^9$	$1.371x10^9$

Table 4.4: Distribution Factor Variation for Change in Girder Spacing for IT 500 on 18.3 m Span

Girder Spacing	610 mm	660 mm	710 mm	735 mm
Shear Distribution Factor	0.348	0.346	0.348	0.363
Moment Distribution Factor	0.187	0.200	0.215	0.218

Table 4.5: Comparison between Distribution Factor Depending on the Position of the Truck when Loaded with Single Truck

	Wheel line on Exterior Beam				Wheel line on First Interior Beam			
Simple Span	**IT 400 (12.2 m SPAN)**							
	Rigid Method	**Grillage**	**LRFD**	**Std. Spec.**	**Grillage**	**1.2*Grillage**	**LRFD**	**Std. Spec.**
Moment	0.179	0.202	0.500	0.184	0.158	0.189	0.209	0.184
Shear	0.179	0.369	0.500	0.184	0.348	0.417	0.440	0.184
Continuous Span								
Positive Moment	0.179	0.217	0.500	0.184	0.167	0.200	0.209	0.184
Shear	0.179	0.385	0.500	0.184	0.342	0.411	0.440	0.184
Negative Moment	0.179	0.261	0.500	0.184	0.170	0.203	0.209	0.184
Simple Span	**IT 500 (18.3 m SPAN)**							
Moment	0.179	0.178	0.500	0.178	0.142	0.170	0.191	0.178
Shear	0.179	0.356	0.500	0.178	0.327	0.393	0.440	0.178
Continuous Span								
Positive Moment	0.179	0.192	0.500	0.178	0.150	0.180	0.191	0.178
Shear	0.179	0.376	0.500	0.178	0.326	0.391	0.440	0.178
Negative Moment	0.179	0.232	0.500	0.178	0.162	0.195	0.191	0.178
Simple Span	**IT 700 (24.4 m SPAN)**							
Moment	0.179	0.182	0.500	0.176	0.144	0.172	0.184	0.176
Shear	0.179	0.358	0.500	0.176	0.321	0.385	0.440	0.176
Continuous Span								
Positive Moment	0.179	0.196	0.500	0.176	0.151	0.181	0.184	0.176
Shear	0.179	0.379	0.500	0.176	0.321	0.385	0.440	0.176
Negative Moment	0.179	0.229	0.500	0.176	0.165	0.197	0.184	0.176
Simple Span	**IT 900 (30.5 m SPAN)**							
Moment	0.179	0.185	0.500	0.176	0.144	0.173	0.180	0.176
Shear	0.179	0.362	0.500	0.176	0.318	0.381	0.440	0.176
Continuous Span								
Positive Moment	0.179	0.198	0.500	0.176	0.151	0.182	0.180	0.176
Shear	0.179	0.383	0.500	0.176	0.320	0.384	0.440	0.176
Negative Moment	0.179	0.230	0.500	0.176	0.172	0.206	0.180	0.176

Table 4.6: Comparison between Distribution Factors Depending on the Position of the Truck when Loaded with Two Trucks

	Wheel line on Exterior Beam				Wheel line on First Interior Beam		
Simple Span	**IT 400 (12.2 m SPAN)**						
	Rigid Method	**Grillage**	**LRFD**	**Std. Spec.**	**Grillage**	**LRFD**	**Std. Spec.**
Moment	0.179	0.241	0.247	0.184	0.197	0.247	0.184
Shear	0.179	0.384	0.363	0.184	0.366	0.363	0.184
Continuous Span							
Positive Moment	0.179	0.250	0.247	0.184	0.203	0.247	0.184
Shear	0.179	0.389	0.363	0.184	0.359	0.363	0.184
Negative Moment	0.179	0.269	0.247	0.184	0.201	0.247	0.184
Simple Span	**IT 500 (18.3 m SPAN)**						
Moment	0.179	0.228	0.232	0.178	0.192	0.232	0.178
Shear	0.179	0.371	0.363	0.178	0.348	0.363	0.178
Continuous Span							
Positive Moment	0.179	0.236	0.232	0.178	0.192	0.232	0.178
Shear	0.179	0.380	0.363	0.178	0.345	0.363	0.178
Negative Moment	0.179	0.250	0.232	0.178	0.189	0.232	0.178
Simple Span	**IT 700 (24.4 m SPAN)**						
Moment	0.179	0.231	0.229	0.176	0.186	0.229	0.176
Shear	0.179	0.370	0.363	0.176	0.341	0.363	0.176
Continuous Span							
Positive Moment	0.179	0.239	0.229	0.176	0.189	0.229	0.176
Shear	0.179	0.381	0.363	0.176	0.340	0.363	0.176
Negative Moment	0.179	0.250	0.229	0.176	0.187	0.229	0.176
Simple Span	**IT 900 (30.5 m SPAN)**						
Moment	0.179	0.234	0.227	0.176	0.186	0.227	0.176
Shear	0.179	0.371	0.363	0.176	0.338	0.363	0.176
Continuous Span							
Positive Moment	0.179	0.241	0.227	0.176	0.188	0.227	0.176
Shear	0.179	0.384	0.363	0.176	0.338	0.363	0.176
Negative Moment	0.179	0.252	0.227	0.176	0.186	0.227	0.176

Table 4.7: Effect of Multiple Presence of Trucks (with Multiplication Factor)

IT 400 (12.2 m SPAN) 11 m wide bridge					
Number of trucks		**Three**	**Two**	**Three**	**Two**
Girder		Exterior	Exterior	First Interior	First Interior
Grillage	Moment	0.208	0.240	0.175	0.191
	Shear	0.328	0.385	0.314	0.363
LRFD	Moment	0.247	0.247	0.247	0.247
	Shear	0.363	0.363	0.363	0.363
Std. Specifications	Moment	0.184	0.184	0.184	0.184
	Shear	0.184	0.184	0.184	0.184
IT 500 (18.3 m SPAN) 11 m wide bridge					
Number of trucks		**Three**	**Two**	**Three**	**Two**
Girder		Exterior	Exterior	First Interior	First Interior
Grillage	Moment	0.199	0.225	0.167	0.182
	Shear	0.316	0.371	0.299	0.345
LRFD	Moment	0.232	0.232	0.232	0.232
	Shear	0.363	0.363	0.363	0.363
Std. Specifications	Moment	0.178	0.178	0.178	0.178
	Shear	0.178	0.178	0.178	0.178
IT 700 (24.4 m SPAN) 11 m wide bridge					
Number of trucks		**Three**	**Two**	**Three**	**Two**
Girder		Exterior	Exterior	First Interior	First Interior
Grillage	Moment	0.202	0.230	0.167	0.184
	Shear	0.314	0.369	0.293	0.338
LRFD	Moment	0.229	0.229	0.229	0.229
	Shear	0.363	0.363	0.363	0.363
Std. Specifications	Moment	0.176	0.176	0.176	0.176
	Shear	0.176	0.176	0.176	0.176
IT 900 (30.5 m SPAN) 11 m wide bridge					
Number of trucks		**Three**	**Two**	**Three**	**Two**
Girder		Exterior	Exterior	First Interior	First Interior
Grillage	Moment	0.203	0.232	0.166	0.185
	Shear	0.315	0.371	0.290	0.336
LRFD	Moment	0.227	0.227	0.227	0.227
	Shear	0.363	0.363	0.363	0.363
Std. Specifications	Moment	0.176	0.176	0.176	0.176
	Shear	0.176	0.176	0.176	0.176

Table 4.8: Effect of Change in Width of Bridge Model with Two Trucks (no Multiple Presence Factor)

		Exterior Girder			
Bridge width		**6.7m**	**11m**	**6.7m**	**11m**
IT 400 (12.2 m Span)	**Flexure**	0.197	0.191	0.241	0.240
	Shear	0.366	0.363	0.384	0.385
IT 500 (18.3 m Span)	**Flexure**	0.192	0.182	0.228	0.225
	Shear	0.348	0.345	0.371	0.371
IT 700 (24.4 m Span)	**Flexure**	0.186	0.184	0.231	0.230
	Shear	0.341	0.338	0.370	0.369
IT 900 (30.5 m Span)	**Flexure**	0.186	0.185	0.234	0.232
	Shear	0.338	0.336	0.371	0.371

Table 4.9 Comparison of Moment Distribution Factors for Skew Bridges (Wheel-Line on the First Interior Girder)

			0	**15**	**30**	**45**	**60**
Interior Girder	**One trucks**	**Grillage**	0.170	0.157	0.135	0.117	0.086
		LRFD	0.191	0.190	0.189	0.186	0.180
		Std. Spec.	0.182	0.182	0.182	0.182	0.182
	Two trucks	**Grillage**	0.187	0.171	0.144	0.113	0.079
		LRFD	0.232	0.231	0.230	0.226	0.219
		Std. Spec.	0.182	0.182	0.182	0.182	0.182
	One trucks	**Grillage**	0.160	0.157	0.138	0.116	0.081
		LRFD	0.191	0.190	0.189	0.186	0.180
		Std. Spec.	0.182	0.182	0.182	0.182	0.182
	One trucks	**Grillage**	0.172	0.170	0.141	0.110	0.072
		LRFD	0.232	0.231	0.230	0.226	0.219
		Std. Spec.	0.182	0.182	0.182	0.182	0.182

Table 4.10: Comparison of Shear Distribution Factors for Skew Bridges (Wheel-Line on the First Interior Girder)

	Skew Angle		0	15	30	45	60
Exterior Girder	One trucks	Grillage	0.393	0.403	0.403	0.397	0.387
		LRFD	0.440	0.488	0.543	0.618	0.748
		Std. Spec.	0.182	0.182	0.182	0.182	0.182
	Two trucks	Grillage	0.348	0.390	0.400	0.394	0.374
		LRFD	0.363	0.403	0.448	0.510	0.618
		Std. Spec.	0.182	0.182	0.182	0.182	0.182
	One trucks	Grillage	0.087	0.434	0.675	0.785	0.773
		LRFD	0.440	0.488	0.543	0.618	0.748
		Std. Spec.	0.182	0.182	0.182	0.182	0.182
	One trucks	Grillage	0.393	0.403	0.403	0.397	0.387
		LRFD	0.440	0.488	0.543	0.618	0.748
		Std. Spec.	0.182	0.182	0.182	0.182	0.182
	Two trucks	Grillage	0.348	0.390	0.400	0.394	0.374
		LRFD	0.363	0.403	0.448	0.510	0.618
		Std. Spec.	0.182	0.182	0.182	0.182	0.182

Note- Grillage analyses values contain multiple presence factor where AASHTO LRFD equations have been directly used, and not for conditions where lever rule is used.

Table 4.11 Variation in Shear Distribution Factor in Exterior Girder for Various Conditions (Wheel-Line on First Interior Girder)

Angle (degrees)	Number of trucks	Without Diaphragm	With Diaphragm	Cracked slab section	Cracked section with diaphragm
0	1 truck	0.087	0.097	0.087	0.101
0	2 trucks	0.082	0.075	0.083	0.060
15	1 truck	0.434	0.330	0.130	0.287
15	2 trucks	0.449	0.154	0.153	0.135
30	1 truck	0.675	0.489	0.529	0.427
30	2 trucks	0.682	0.247	0.536	0.210
45	1 truck	0.785	0.598	0.646	0.581
45	2 trucks	0.750	0.429	0.621	0.355
60	1 truck	0.773	0.840	0.670	0.728
60	2 trucks	0.693	0.673	0.605	0.553

Table 4.12 Variation in Shear Distribution Factor in Interior Girder for various Conditions (wheel-line on First Interior Girder)

Angle (degrees)	Number of trucks	Without diaphragm	With diaphragm	Cracked slab section	Cracked section with diaphragm
0	**1 truck**	0.393	0.398	0.391	0.413
0	**2 trucks**	0.348	0.354	0.347	0.361
15	**1 truck**	0.403	0.376	0.403	0.395
15	**2 trucks**	0.390	0.396	0.381	0.394
30	**1 truck**	0.403	0.341	0.410	0.369
30	**2 trucks**	0.400	0.401	0.396	0.401
45	**1 truck**	0.397	0.295	0.412	0.337
45	**2 trucks**	0.394	0.363	0.397	0.381
60	**1 truck**	0.387	0.246	0.407	0.302
60	**2 trucks**	0.374	0.272	0.382	0.321

Note- No multiplication factor is used in the above comparison.

Table 4.13 Comparison of Maximum Girder Reaction for Finite Element Analysis with Grillage Analogy for Various Boundary Conditions

Skew	Truck description	2-D Grillage1	Finite Element1	2-D Grillage2	Finite Element2	Finite Element3
						Moment Y fixed
	EXTERIOR					
0 degree	Two trucks	126.9	132.3	148.8	148.05	143.55
0 degree	One truck	122.0	130.5	159.6	156.15	148.5
15 degree	Two trucks	207.9	243.0	150.6	155.25	148.95
15 degree	One truck	181.4	204.3	167.2	167.40	158.4
30 degree	Two trucks	294.5	318.5	143.7	139.50	127.0
30 degree	One truck	239.1	266.7	159.4	156.15	135.9
45 degree	Two trucks	334.6	340.3	183.3	174.15	153.0
45 degree	One truck	273.2	292.9	192.3	184.95	158.9
60 degree	Two trucks	322.2	312.8	237.2	227.25	180.6
60 degree	One truck	280.2	279.6	235.7	225.90	184.1
	INTERIOR					
0 degree	Two trucks	119.25	120.2	121.8	121.1	122.0
0 degree	One truck	112.1	113.4	112.1	111.6	122.0
15 degree	Two trucks	153.45	170.6	53.7	52.2	49.1
15 degree	One truck	124.2	145.8	73.9	74.7	59.0
30 degree	Two trucks	233.2	239.1	70.4	66.7	63.5
30 degree	One truck	192.4	207.6	92.7	90.0	73.8
45 degree	Two trucks	256.6	251.1	99.5	93.6	83.0
45 degree	One truck	223.7	224.4	116.7	111.6	91.8
60 degree	Two trucks	237.1	225.7	144.4	136.8	108.4
60 degree	One truck	220.3	208.8	153.0	144.5	114.3

Note:-
1 – Pinned for bending, fixed for torsion
2 – Pinned for bending, released for torsion.
3 – Fixed for bending, fixed for torsion.
Trucks are placed in such a manner that one of the wheel-line lied on the particular girder under consideration. All values are in KN. No Multiplication factor is used.

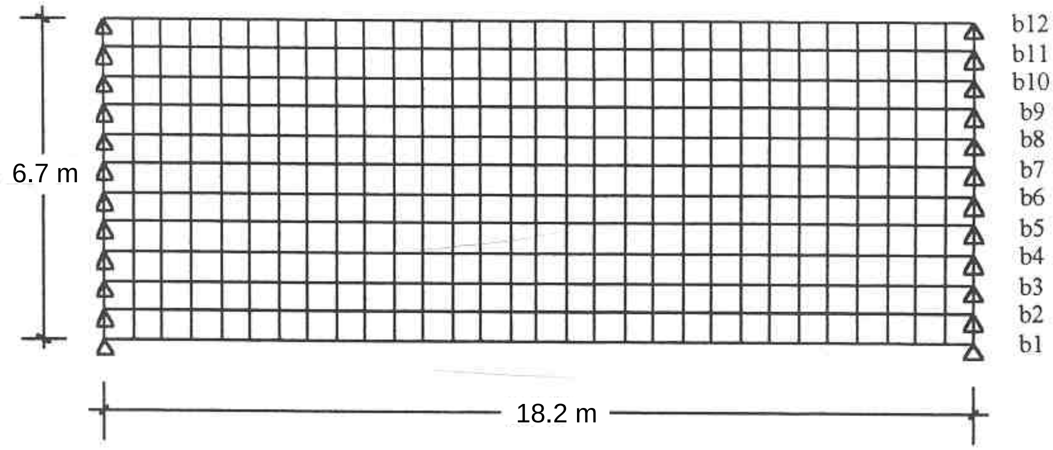


Figure 4.1: Member Discretization of 6.7-meter Wide Bridge Model on 18.2-meter Span

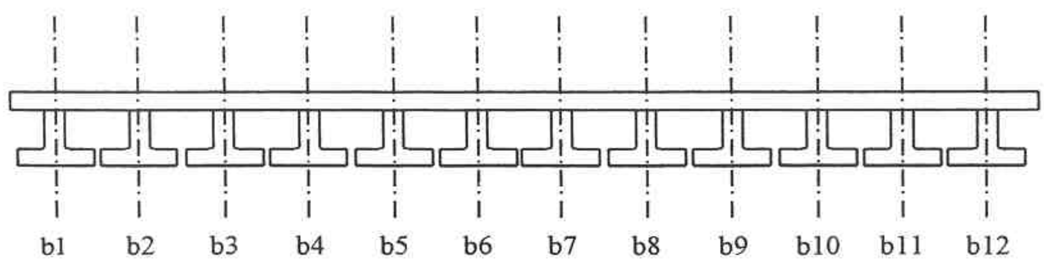


Figure 4.2: Idealized Cross-Section

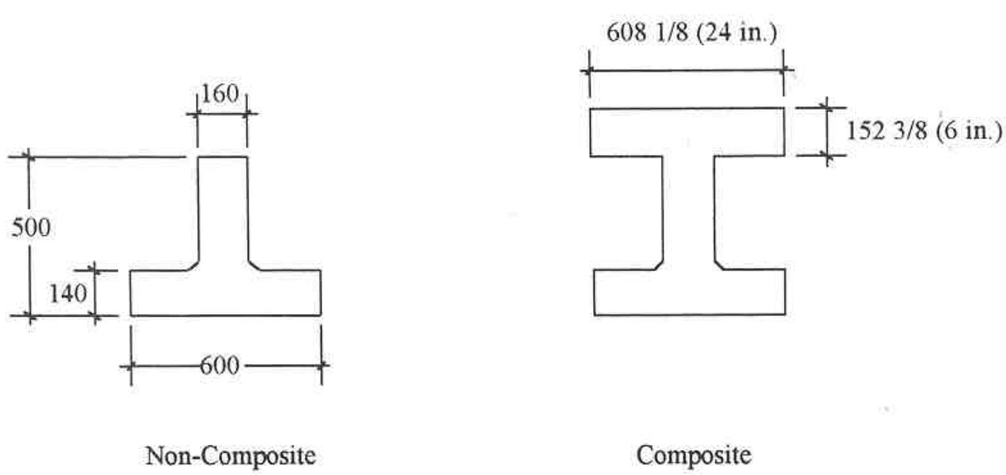


Figure 4.3: Cross-Section of IT-500

(Note all dimensions are in mm unless noted)

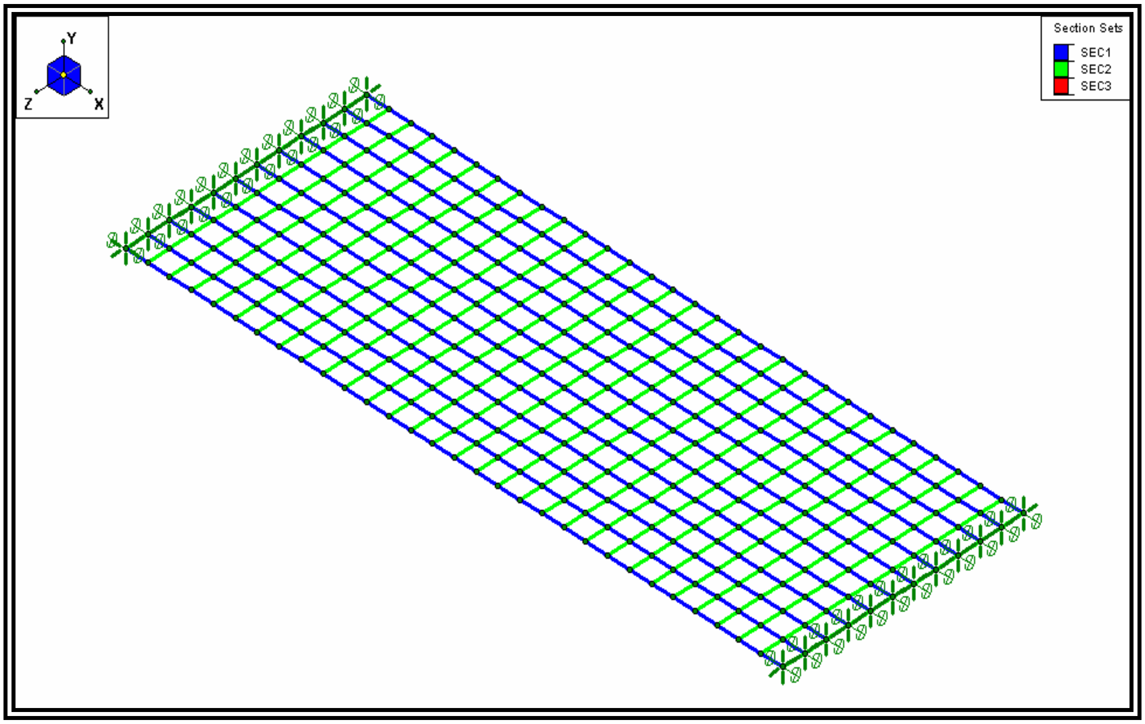


Figure 4.4: Typical Grillage Analogy Model of a Non-skew bridge

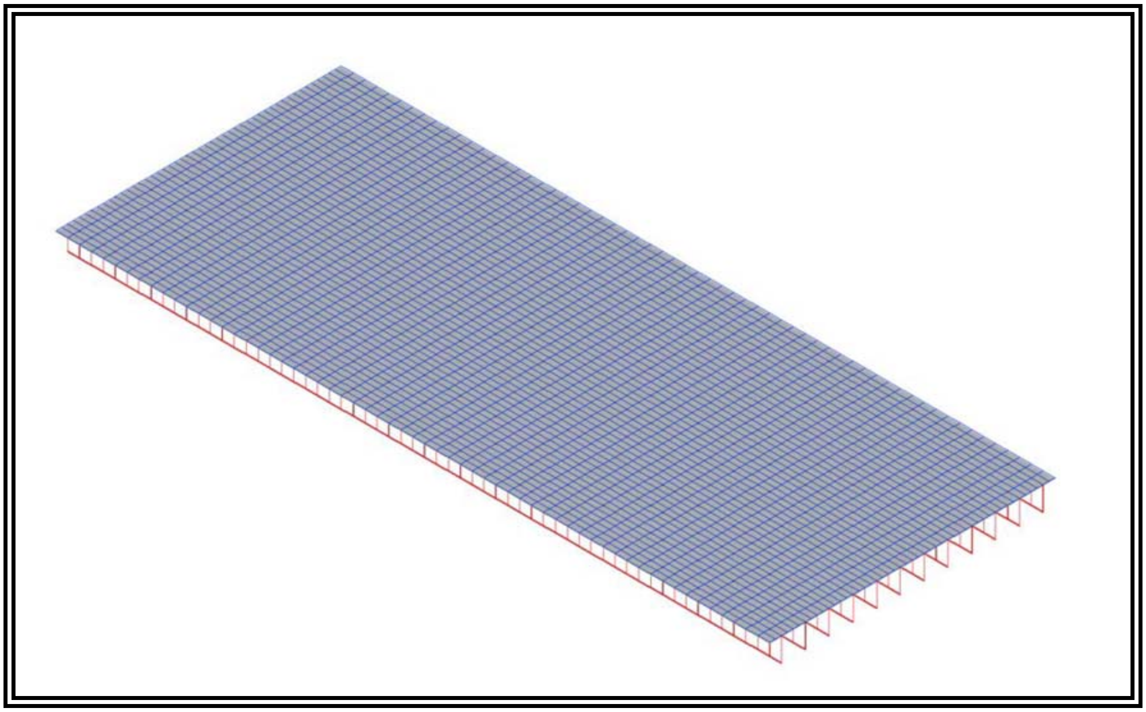


Figure 4.5: Typical Finite Element Analysis Model of a Non Skew bridge

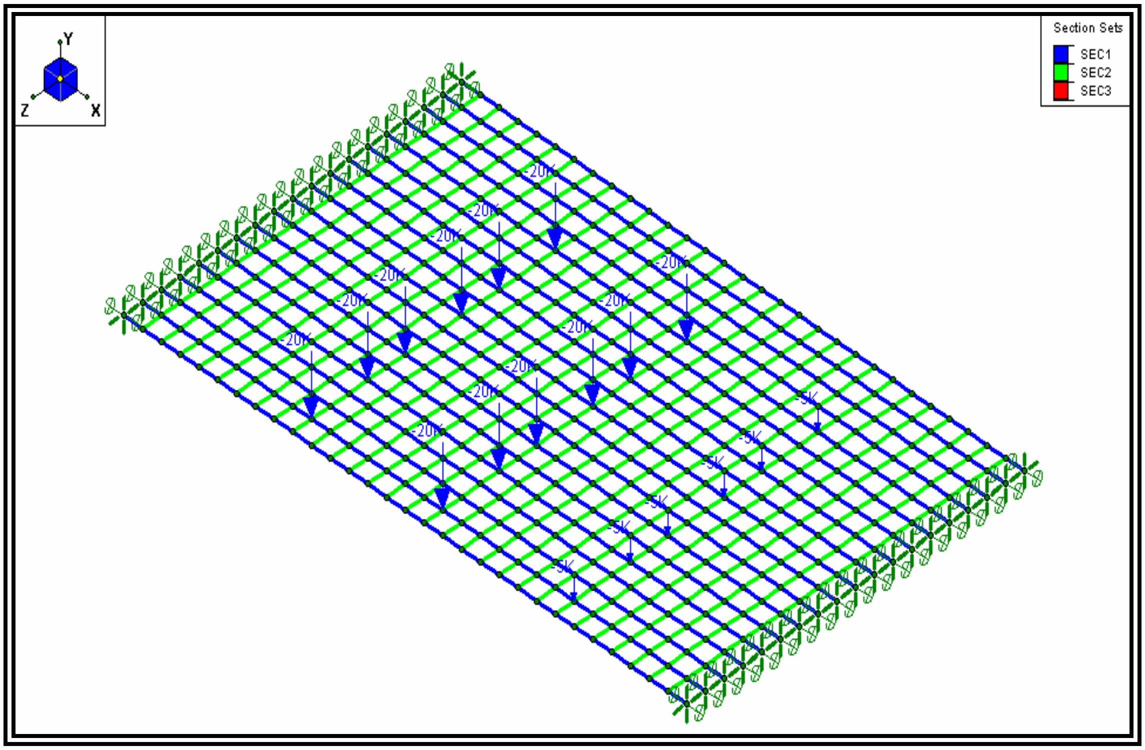


Figure 4.6: Typical Grillage Analogy Model of a 11-meter wide Bridge with three trucks loaded case

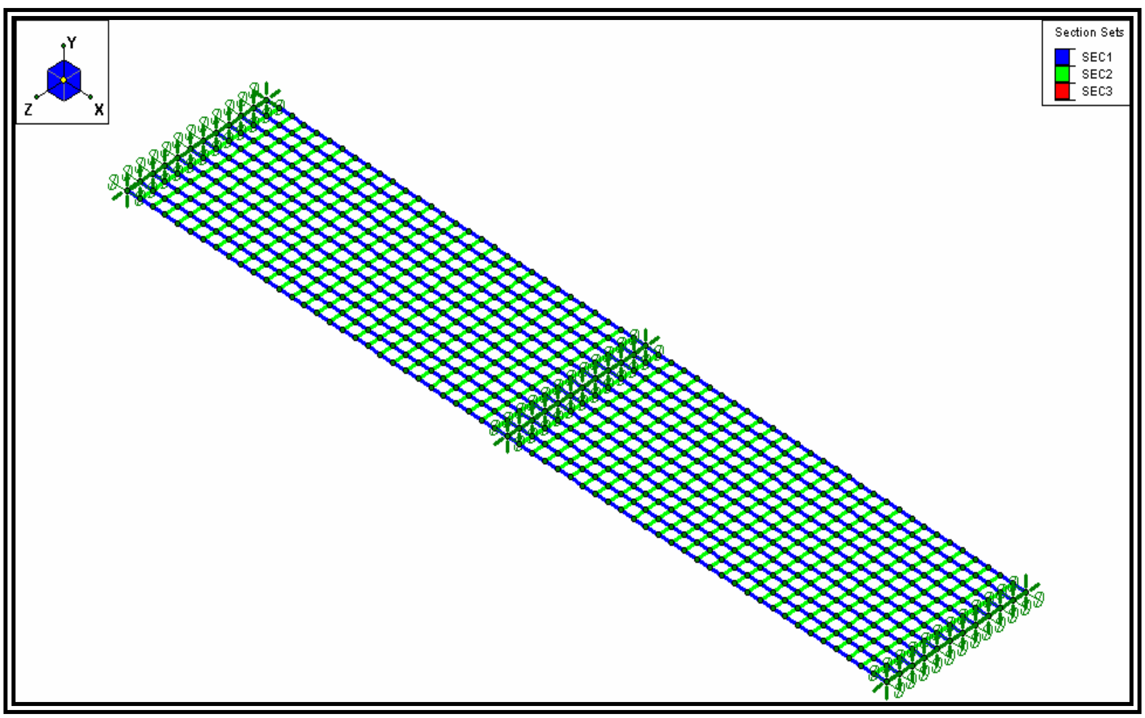
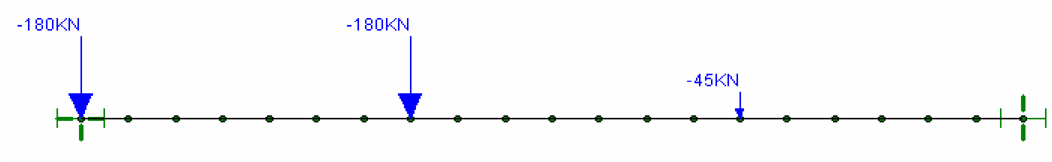
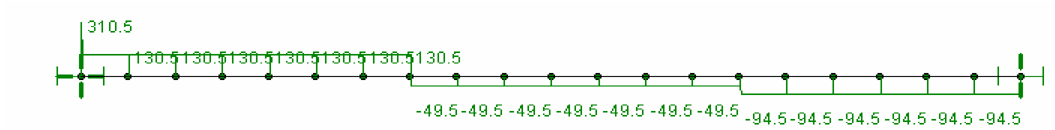


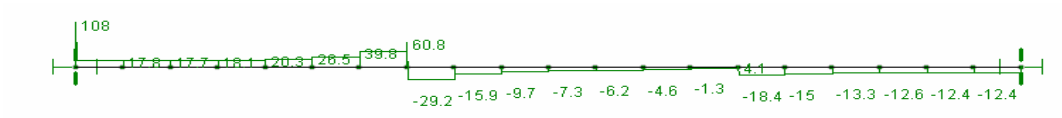
Figure 4.7: Typical Grillage Analogy Model of Two Span Continuous Bridge



HS-25 Truck position for Maximum Shear

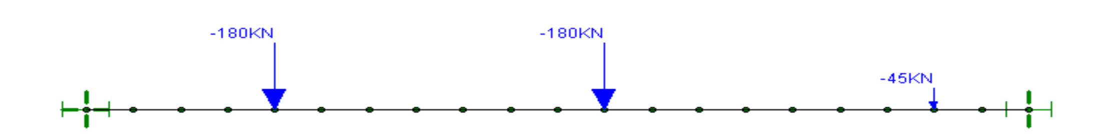


Maximum Shear Diagram for HS-25

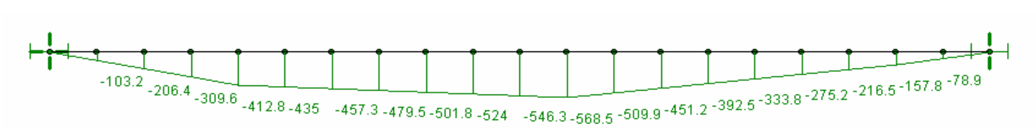


Shear Force Diagram of beam b2 when the truck is placed on the first interior girder for one truck loaded case (Grid Analysis result for IT 400 12.1 m span)

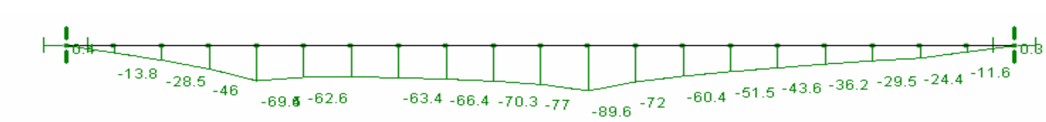
Figure 4.8: Determination of Live Load Shear Distribution Factor for IT 400.



HS-25 Truck position for Maximum Moment



Maximum Moment Diagram for HS-25



Moment Force Diagram of beam b2 when the truck is placed on the first interior girder for one truck loaded case (Grid Analysis result for IT 400 12.1 m span)

Figure 4.9: Determination of Live Load Moment Distribution Factor for IT 400

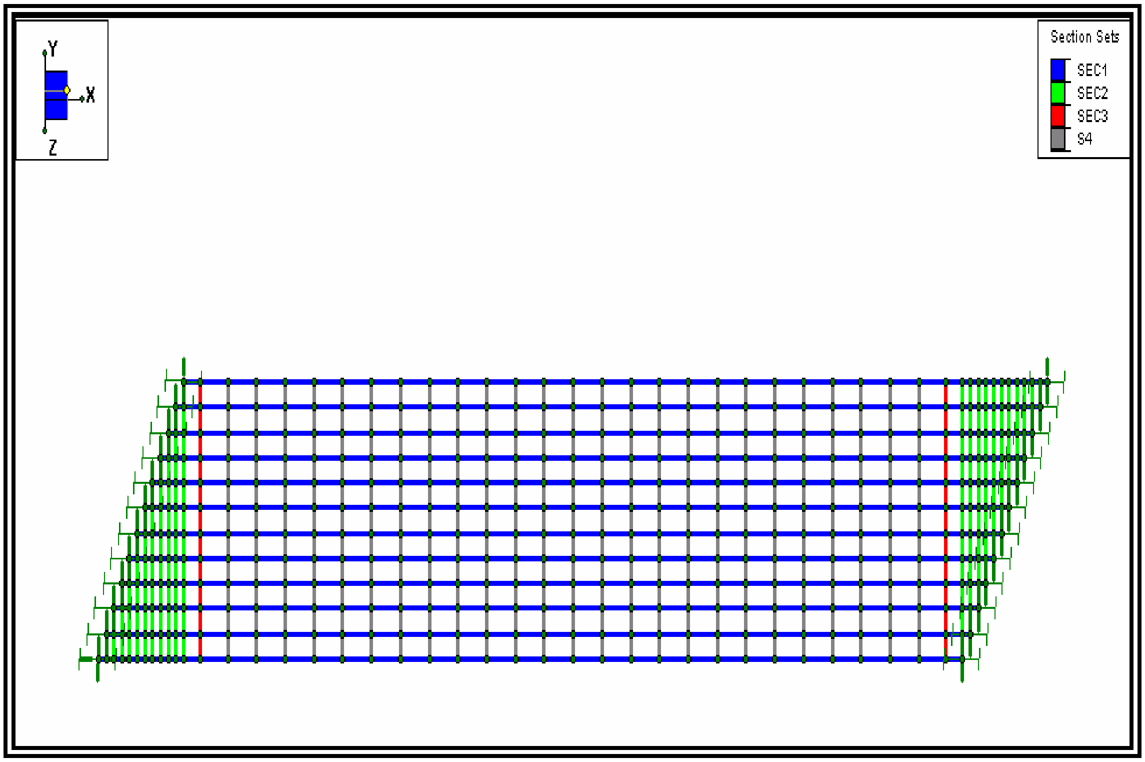


Figure 4.10: Typical Grillage Analogy Model of a Skew Bridge

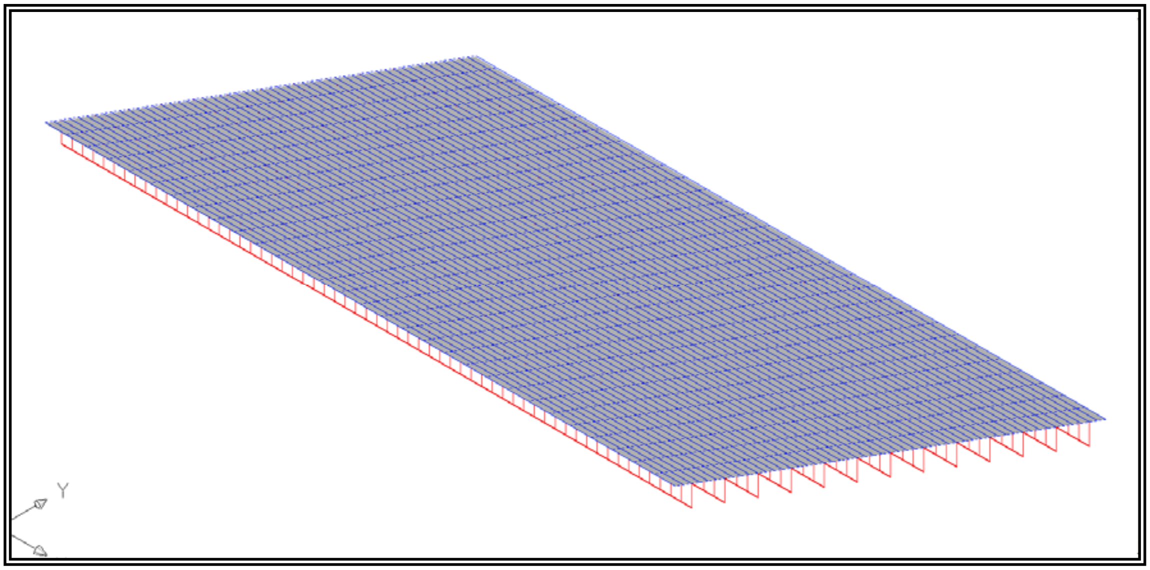


Figure 4.11: Typical Finite Element Analysis Model of a Skew Bridge

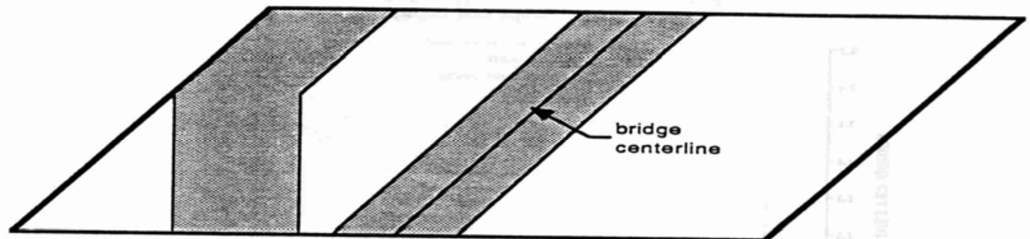


FIGURE E.1: The General Space in Which Trucks are Placed to Maximize Girder Response in Skewed Bridges.

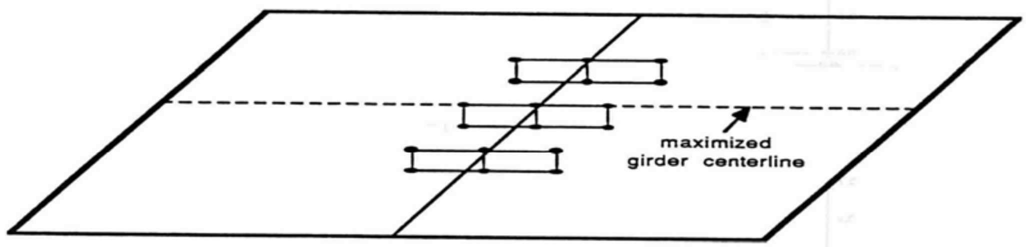


FIGURE E.2: General Truck Placement to Maximize Girder Moment Response in Skewed Bridges.

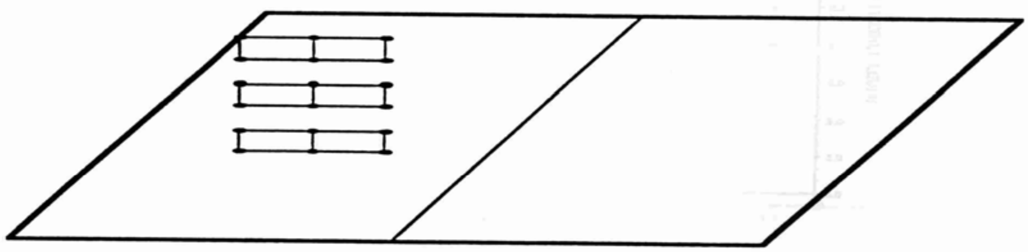


FIGURE E.3: General Truck Placement to Maximize Shear in the Obtuse Corner Girder of Skewed Bridges.

Figure 4.12: Truck positions as per NCHRP Report

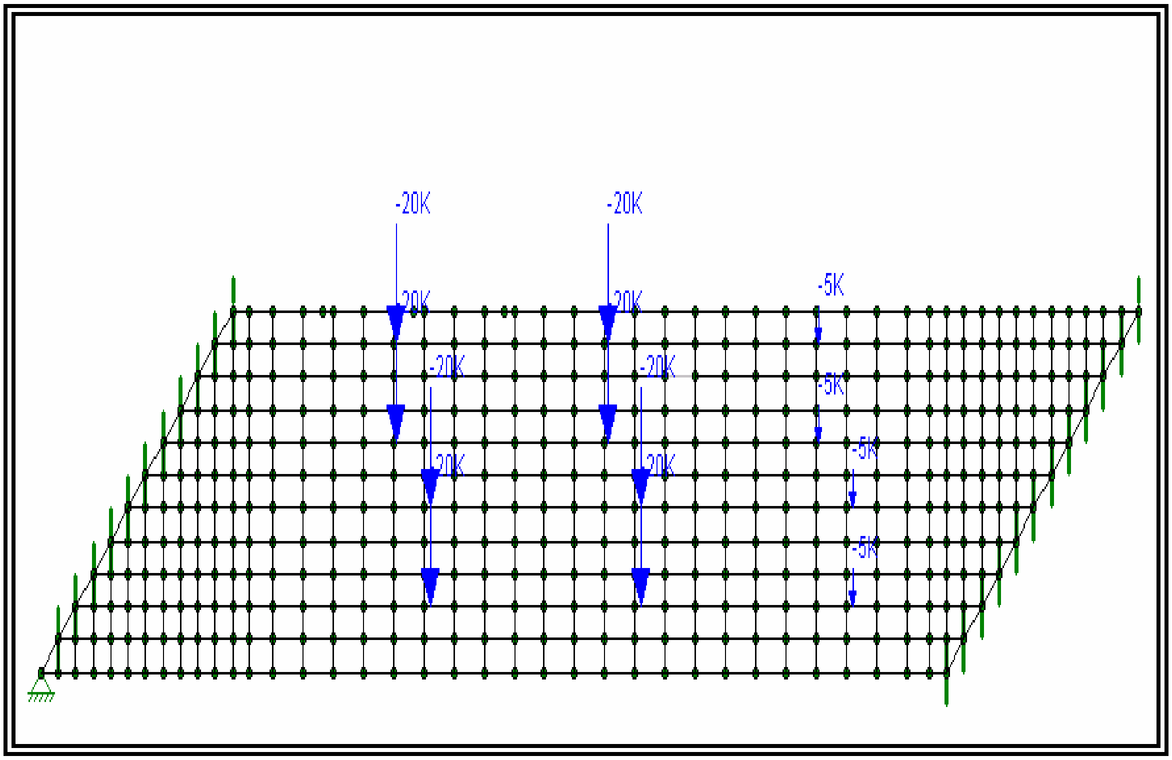


Figure 4.13: Actual truck position for Shear in Interior Girder

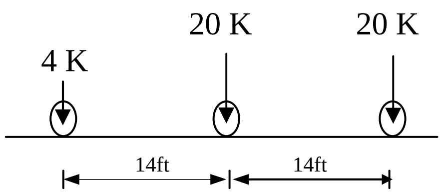


Figure 4.14: Truck model HS-25

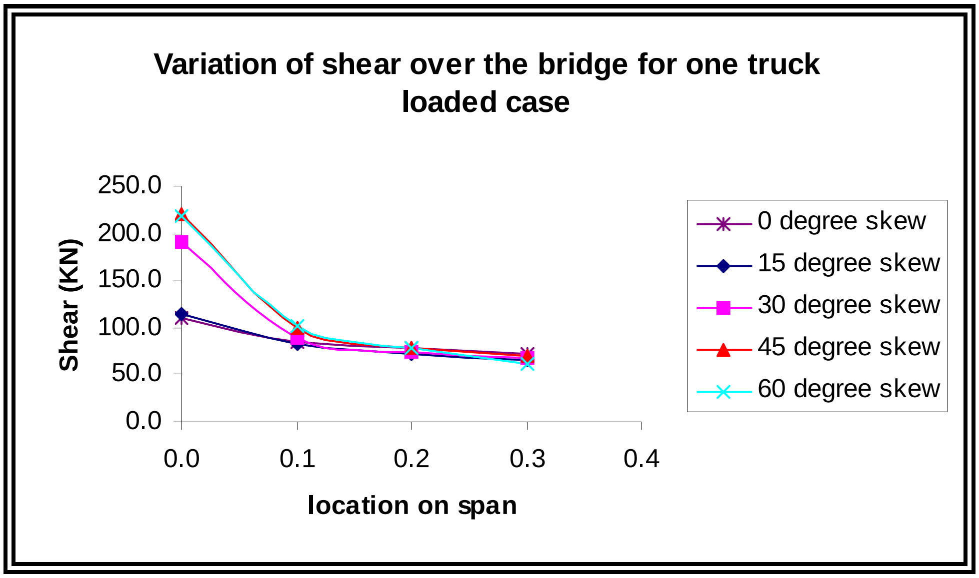


Figure 4.15: Shear Variation Along the Span for One Truck Loaded Case

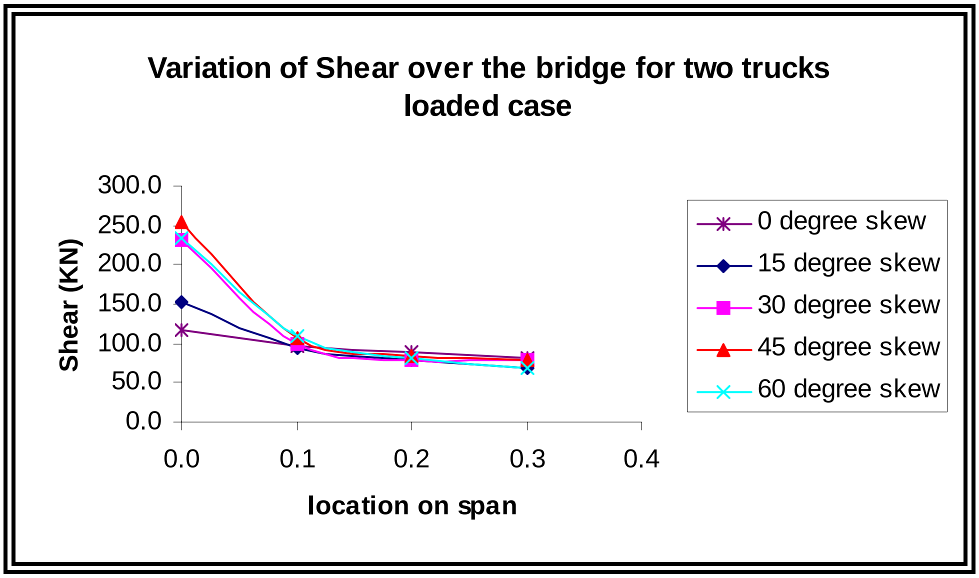


Figure 4.16: Shear Variation Along the Span for Two Trucks Loaded Case

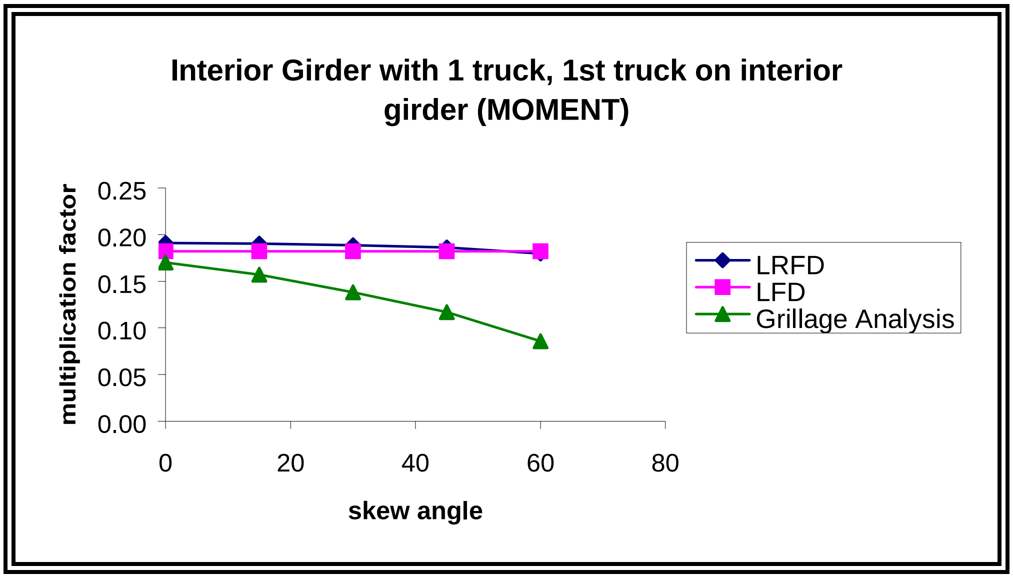


Figure 4.17: Interior Girder Moment Distribution with One Truck, Wheel-Line on Interior Girder

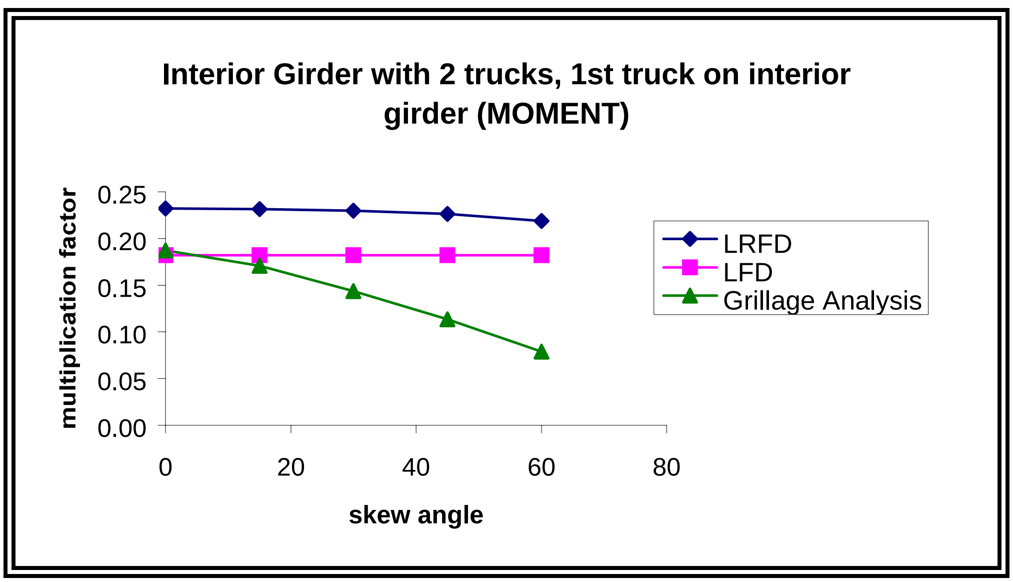


Figure 4.18: Interior Girder Moment Distribution with Two Trucks, Wheel-Line on Interior Girder

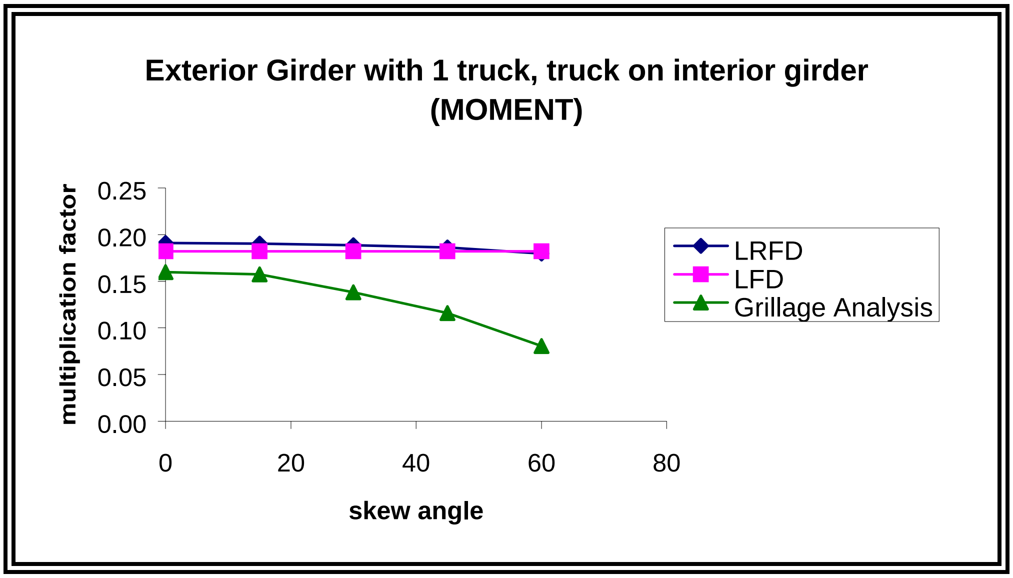


Figure 4.19: Exterior Girder Moment Distribution with One Truck, Wheel-Line on Interior Girder

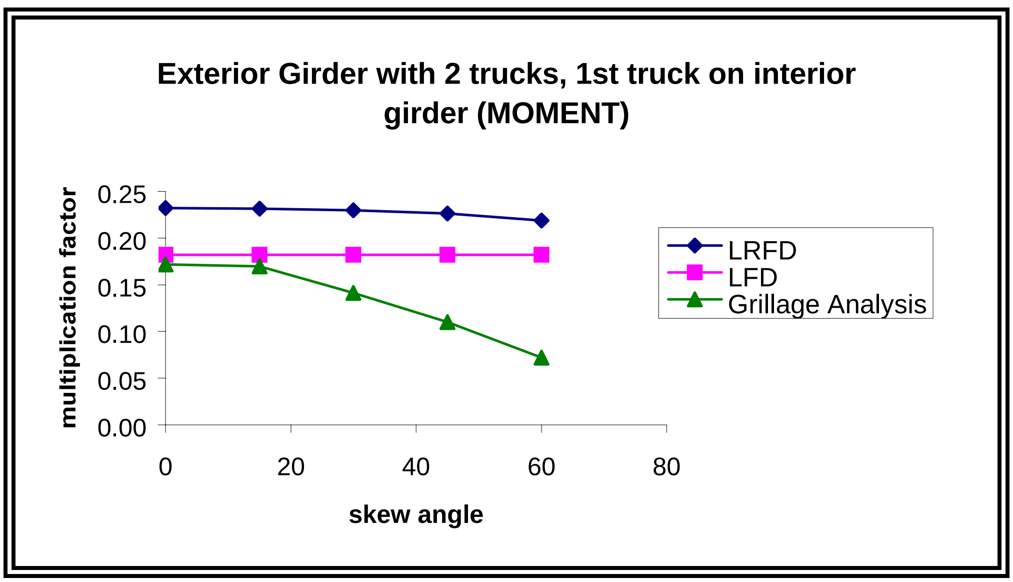


Figure 4.20: Exterior Girder Moment Distribution with Two Trucks, Wheel-Line on Interior Girder

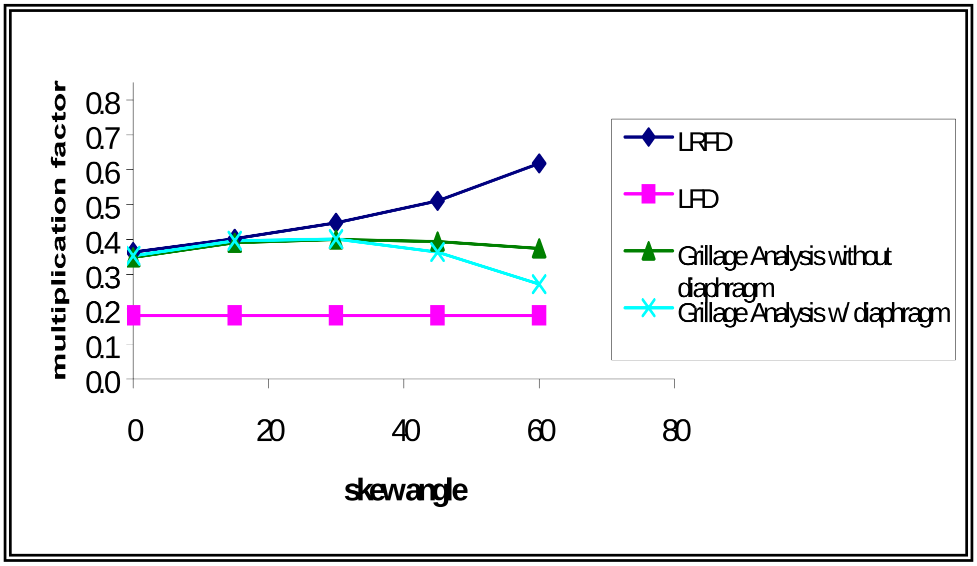


Figure 4.21: Interior Girder Shear Distribution with One Truck, Wheel-Line on Interior Girder

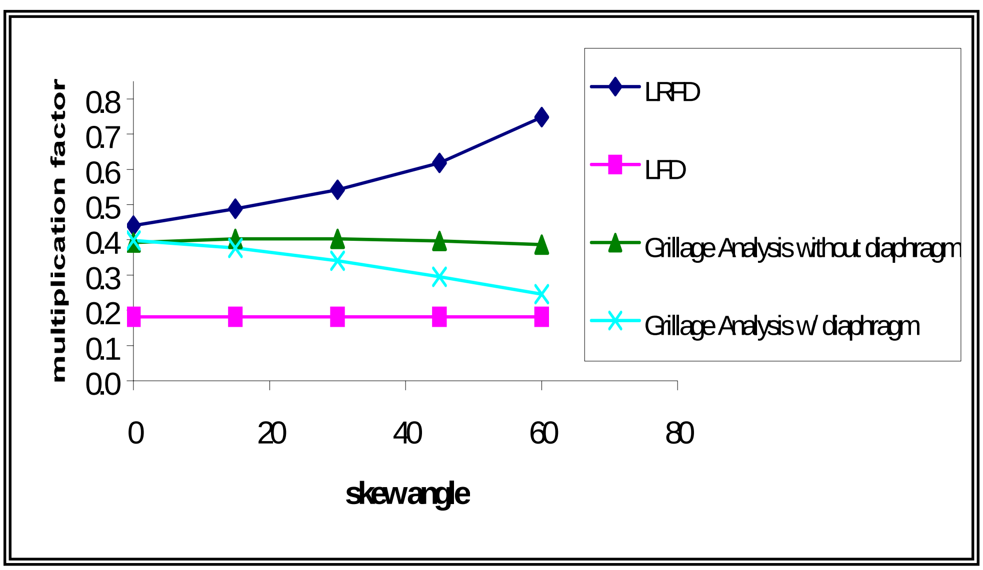


Figure 4.22: Interior Girder Shear Distribution with Two Trucks, Wheel-Line on Interior Girder

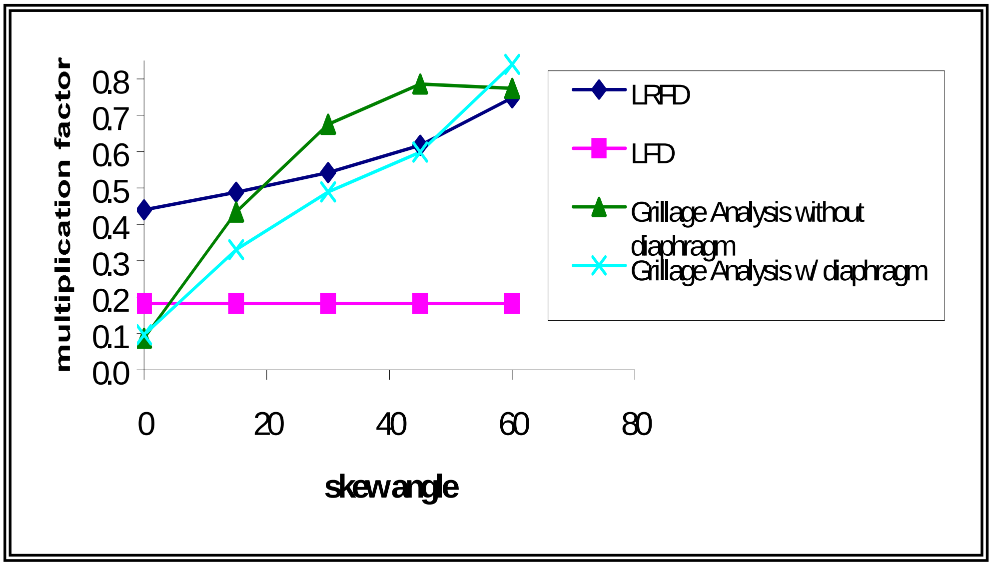


Figure 4.23: Exterior Girder Shear Distribution with One Truck, Wheel-Line on Interior Girder

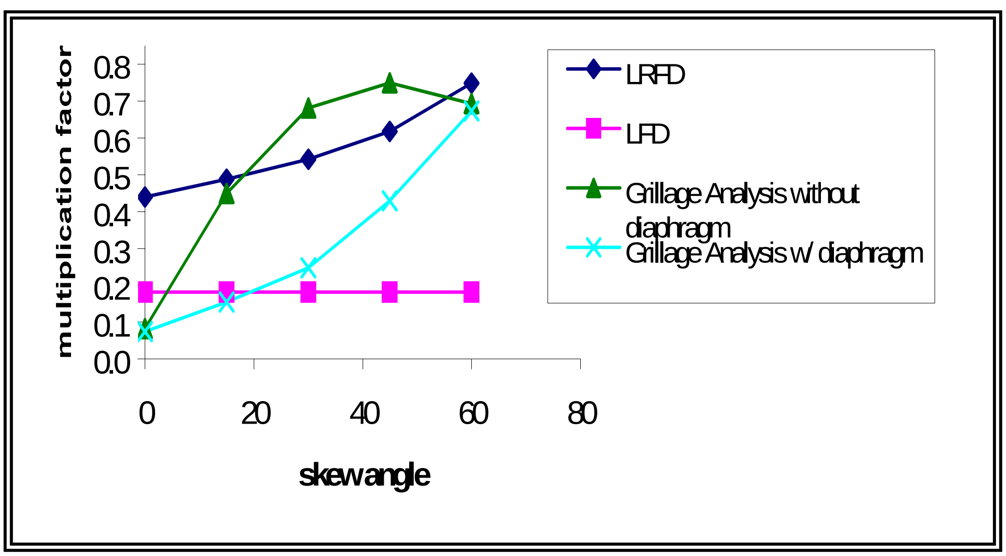


Figure 4.24: Exterior Girder Shear Distribution with Two Trucks, Wheel-Line on Interior Girder

Chapter 5

Design of Inverted Tee Girders

5.1 Introduction

Designing the IT girders was also an important part of the investigation. This process included the use of the live load distribution factors derived earlier. The designs involved multiple analyses on bridge models with varying parameters including span lengths, number of spans, widths and skew angles.

5.2 Prototypes of bridges

There were six prototype bridges that were designed. These are shown below:

Bridge Type	Span (m)	Width (m)	Skew/ Non Skew
Single Span	8	8.5	Both
Single Span	16	11	Both
Single Span	24	12	Non Skew
Continuous Span	21 - 26.25 - 21	8	Non Skew
Continuous Span	42 - 52.5 - 42	11	Non Skew
Continuous Span	63 - 78.75 - 63	12	Non Skew

5.3 Live Load Cases

The above prototype bridges were designed for three different live load conditions. This was done in order to determine if the loading requirements would result in significantly different structures, such that the development of separate standards would be warranted. These load cases were the following...

(1) KDOT MS-18 load case – This is similar to the live load provision given in the AASHTO Standard Specifications which is the HS20-44 Loading. It consisted of a tractor with a semi-trailer or its corresponding lane load (AASHTO Standard Specifications Article 3.7.6)

(2) KDOT MS-22.5 load case – This is obtained by increasing the MS-18 Truck and MS-18 lane loadings by 25% and is often referred to as HS-25.

(3) HL-93 load case – This is the same live load provision given in AASHTO LRFD which consists of lane loading with a truck (HS20-44) or tandem or a truck train (with 90% effect, for continuous span), giving maximized effect. This is given in Article 3.6.1.1 of AASHTO LRFD.

5.4 Design Requirements

The required design conditions are detailed below [7]

5.4.1 MS-18 and MS-22.5 Loading Condition

Temporary allowable concrete Compressive stresses before loss due to creep and shrinkage is $0.6f'_{ci}$ MPa

Temporary allowable Tension stresses before loss due to creep and shrinkage is $0.25\sqrt{f_{ci}}$ MPa

Allowable Working Stresses in Prestressed beams due to the prestressing force service loads and prestress losses shall be limited to

Compression	$0.4\ f'_{ci}$ MPa
Tension, precompressed tensile zone:	
MS18 Design	0.0 Mpa
Kansas Overload Design (1.25 MS18)	$0.125\sqrt{f_c}$ MPa

Stresses in the concrete at service load (including the future wearing surface) after all prestress losses and additional creep and shrinkage losses caused by the positive moment connection shall be…

Compression: $0.40\ f'_{ci}$ MPa

Tension, precompressed tensile zone: $0.25\sqrt{f_{ci}}$ MPa **

** (applies to both the MS18 Design and the Kansas Overload Design)

5.4.2 HL-93 Loading Conditions

[2] The various stress limits for concrete were as per Article 5.9.4 of AASHTO LRFD

For Temporary Stresses before Losses,

Compressive Stress limit in concrete shall be $0.6f'_{ci}$ MPa.

Tensile Stress limit in concrete in areas other than the precompressed tensile zones and bonded auxiliary reinforcement shall be $0.25\sqrt{f_{ci}}$ MPa [1.38 MPa.

For Stresses at Service Limit State after Losses,

Compressive Stress limit in concrete in other than segmentally constructed bridges due to the sum of effective prestress and permanent loads shall be $0.45f'_c$ MPa.

Compressive Stress limit in concrete in other than segmentally constructed bridges due to due to live load and one-half the sum of the effective prestress and permanent loads shall be $0.4f'_c$ MPa.

Compressive Stress limit in concrete due to sum of effective prestress, permanent loads, and transient loads and during shipping and handling shall be $0.6f'_c$ MPa.

Tension Stress limit in concrete for components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions shall be $0.25\sqrt{f_c}$ MPa.

5.4 Miscellaneous Data

Concrete Properties

f'_c = 55 MPa (girder)

f'_{ci}= 41 MPa (girder)

f'_c = 27.5 MPa (deck slab)

Prestressed Strands used – ½"dia. Low Relaxation Strands with F_{pu} = 270 ksi

Jacking Stress ratio= 0.75

Relative Humidity = 65%

Typical Template for the strands for IT girder - The typical template consists of two rows of 11 straight stands each in the bottom flange and a row of two strands at the top (see figure 5.1). The bottom row at 50 mm from the bottom and the second row at 50 mm on center from the first row. The row with two strands is at a distance of 50 mm for the top of any IT.

5.5 Results

An extensive study was carried out to determine the maximum spans of the inverted tee system. This was done using the HL-93 loading case for AASHTO LRFD stress limit conditions for simple-span bridges only. Figure 5.2 shows the maximum span that a particular IT girder can be used on, based on the stress conditions discussed above.

Table 5.1 shows the strand requirement (for the above bridge prototypes) for the different loading cases. It also shows the different IT girders used for the different spans.

From ultimate moment capacity requirement, all three loading cases required almost the same number of strands.

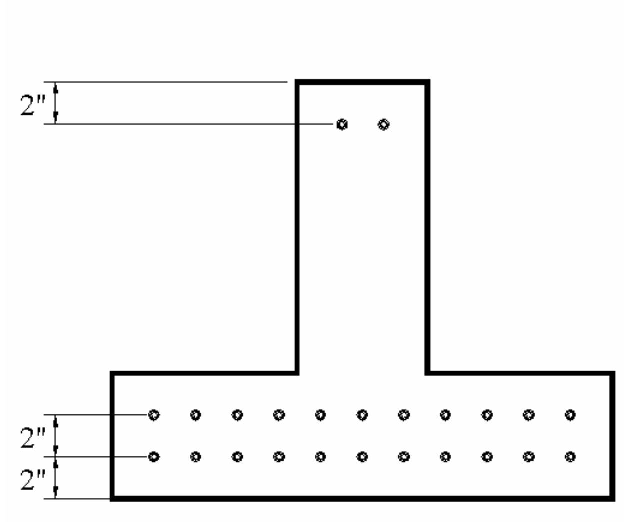


Figure 5.1: Typical Template for Strands for IT Girder
(at ends and mid-span)

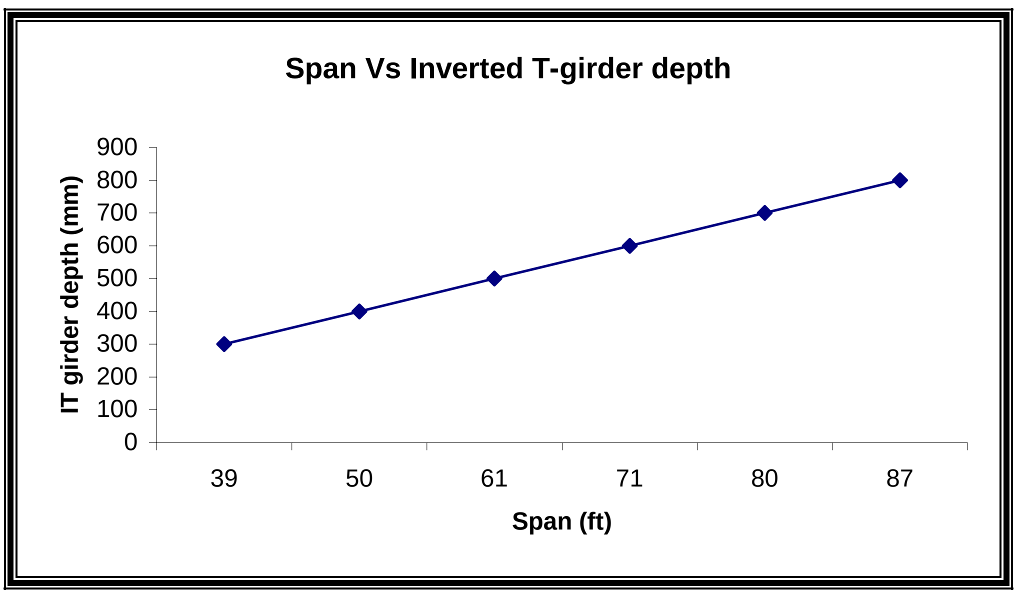


Figure 5.2: Maximum Spans for Inverted Tee System based on HL-93 Loading

Table 5.1: Strand Requirements for Various Spans for IT Beams

Span	Strand Requirement (number)						
	Ultimate Moment Capacity			Stress Requirement			IT used
	MS-18	MS-22.5	HL-93	MS-18	MS-22.5	HL-93	
8m simple	4	4	4	6	8	8	IT300
8m simple (30 skew)	4	4	4	6	8	8	IT300
6.4m-8m-6.4m cont.	3	3	3	8	11	7	IT300
16m simple	8	8	9	14	16	14	IT500
16m simple (30 skew)	8	8	9	14	16	14	IT500
12.8m-16m-12.8m cont.	6	7	6	14	-	13	IT500
24m simple	11	13	12	20	21	19	IT800
19.2m-24m-19.2m cont.	8	9	9	-	-	-	IT800

Chapter 6

Conclusion and Recommendations

6.1 Conclusions

The following main conclusions can be drawn from the investigation of IT bridges conducted as part of this study.

- The AASHTO Standard Specifications (16th Edition) approximate equations give moment live load distribution factors that are generally close to the ones obtained using refined methods, but the shear distribution factors are usually less than half the values obtained using the refined methods.
- AASHTO LRFD (2nd Edition) approximate equations gave live load distribution factor values that were higher than those obtained by refined methods for moment. For shear, the LRFD equations are generally conservative but may be un-conservative at large skew angles.
- The two refined methods gave close results and either of them can be used to model the IT bridges to find the live load distribution.
- The moment and shear distribution factors do not change much when the girder size is changed.
- Increasing in the width of the bridge will not increase the distribution factors if the multiple presence factors given in the AASHTO LRFD (2nd Edition) are used.

6.2 Recommendations

Based on the analyses the following recommendations are made for the inverted tee girder bridge system.

(1) Moment Distribution Factors - It can be seen from the Figures 4.17 to 4.20 that the moment distribution factors for straight bridges (obtained from grillage analysis) are always less than the values obtained using the AASHTO

LRFD approximate equations and the AASHTO LFD formulae for both the interior and exterior girders These figures also shows that for bridges with large skew angles the AASHTO LRFD approximate equations would give over-conservative results for both the one truck loaded and two or more trucks loaded cases. Thus, either the AASHTO LRFD Approximate equations or the AASHTO Standard Specification formula (s/5.5) can be safely used for girders spaced at 2 ft on center. For simplicity, a moment-distribution factor of 0.2 is recommended as shown in Figure 6.1.

(2) Shear Distribution Factors – Considerable time was spent investigating the shear distribution factors for girders in skew bridges. The findings are summarized in the following recommendations.

Interior Girder Shear Distribution Factor

Shear distribution factors obtained from grillage analyses for interior girders were always less than that obtained using AASHTO LRFD, but at the same time greater than those obtained using the AASHTO Standard Specification provisions. Modeling the IT bridges with different end restraints and slab stiffnesses had a considerable impact on shear distribution factors in skewed bridges. However, these values were always less than those computed using the AASHTO LRFD equations. Therefore, a distribution factor value of 0.42 can be safely used for all interior girders as shown in Figure 6.2

Exterior Girder Shear Distribution Factor

It can be seen from Figure 4.23 and 4.24 that in the case of straight bridge (i.e. 0 degree skew) shear distribution factors obtained from grillage analysis (for the exterior girder) were less than those obtained from using the AASHTO LRFD and AASHTO Standard Specification expressions. The grillage models without the end diaphragms give shear values higher than those with the diaphragm. Also, the models with the end diaphragm indicated values comparable to those obtained using AASHTO LRFD. The AASHTO Standard Specification values were far below the values obtained using the grillage models. For skew angles below 20 degrees, a distribution factor value of 0.42 can be used (as per AASHTO LRFD recommendations, exterior girder distribution factor values cannot be less than the values for interior girder). For skew angles greater than 20 degrees the following equation may be used

$$0.42 + \frac{skew\ angle - 20}{100}, \quad \text{where skew angle is in degrees}$$

(See Figure 6.3)

(3) The barrier rails should be placed directly over the exterior girder so that, by eliminating the overhang width, a uniform strand pattern for both interior and exterior beams can be maintained.

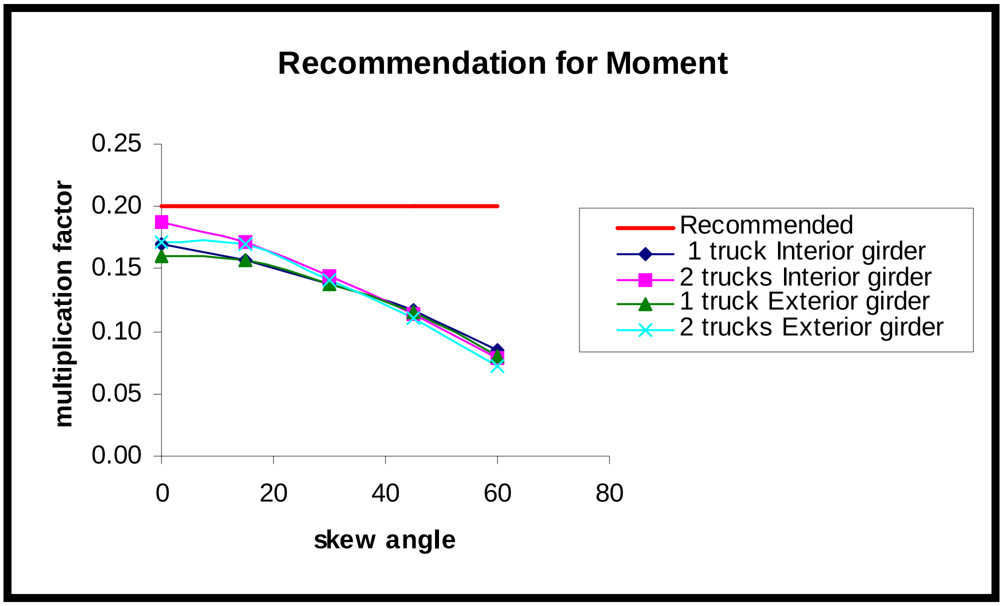


Figure 6.1: Typical Response for Moment Distribution Factors

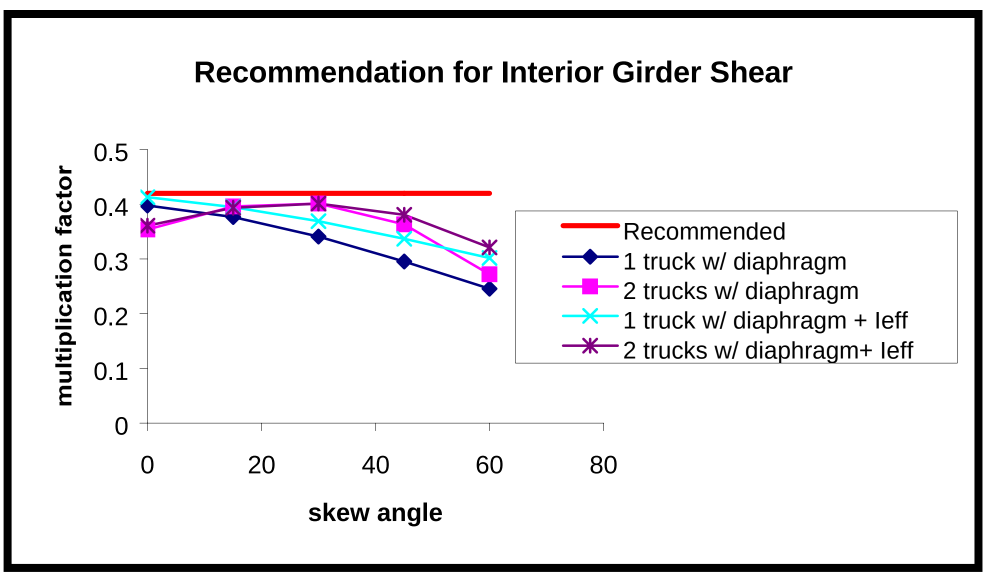


Figure 6.2: Recommended Shear Distribution Factor for Interior Girder

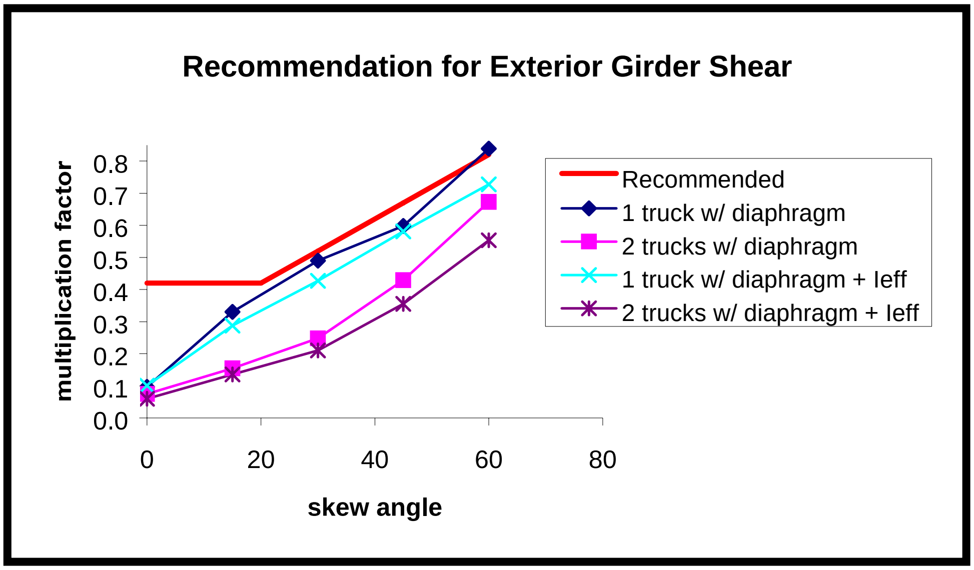


Figure 6.3: Recommended Shear Distribution Factor for Exterior Girder

REFERENCES

(1) American Association of State highway and Transportation Officials (AASHTO). (1996). Standard specifications for highway bridges, Washington, D.C.

(2) American Association of State highway and Transportation Officials (AASHTO). (1998). LRFD bridge design specifications, Washington, D.C.

(3) Tadros, M. K., and Kamel, M. R., “The Inverted Tee Shallow Bridge System for Rural Areas” (September-October 1996), PCI Journal, Vol.41, No.5.

(4) Zokaie, T.,Osterkamp, T.A. and Imbsen, R.A.(1991), “Distribution of wheel loads on Highway bridges”, NCHRP 12-26/1 Final Rep., National Cooperative Highway Research Program, Washington, D.C.

(5) Bishara, A. G., Liu, M. C., and Ali, N.D., “Wheel load Distribution on Simply supported Skew I-beam Composite Bridges”(1992), Journal of Structural Engineering.

(6) Barker, R. M., and Puckett, J. A., (1997) “Design of Highway Bridges”, A Wiley-Interscience Publication.

(7) Kansas Department of Transportation Bridge Design Manual.

Appendix A

Development of a New IT Section with Tapered Flanges

A1.1 Need for a new shape

The bottom flanges of the inverted tee (IT) shape, developed by Dr. Tadros and currently used by the Nebraska Department of Roads, have a horizontal (flat) upper surface. These flat surfaces often trap air pockets during the casting process and result in large "bug holes" on the finished concrete surface. Therefore, one of the objectives added during the course of the research program was to investigate the use of a sloped flange surface to greatly reduce the amount of "bug holes" and to add to the aesthetics of the current block-like shape.

A1.2 Development of New Cross-Section

In the development of a new cross-section with tapered flanges, the following two conditions were established were established.

(1) To identify and utilize a top slope similar to girders in other states.

(2) To maintain the same overall height and width of the IT shape(s), and also to try to match (as close as possible) the section properties of the existing IT shape(s). This would then allow for Kansas precasters to utilize the tapered shape as an alternative to the existing IT shape when bidding jobs in other surrounding states.

An investigation was conducted to determine the bottom-flange slopes currently used by bridge sections in other states. This investigation found that a slope of approximately 17 degrees is common for several other bridge girders, including the Nebraska NU shapes and the Florida Bulb Tee Shape. Thus a slope of 17 degrees was also used for the new Kansas IT Shape. Figures A1 and A2 show the effect of the 17 degree slope on the new shape. The remaining figures show the section properties for both the new shape and the existing shape.

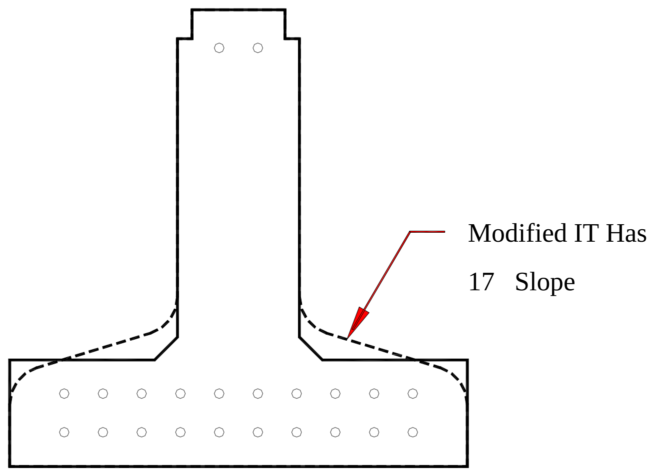


Figure A1: The New IT Shape has a 17° Slope on the Bottom Flange

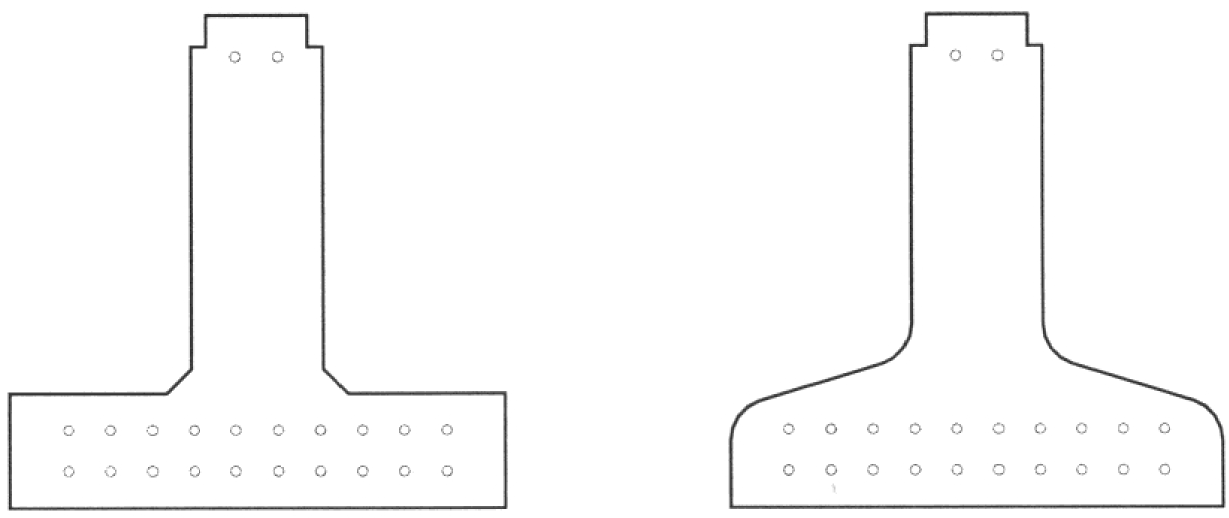


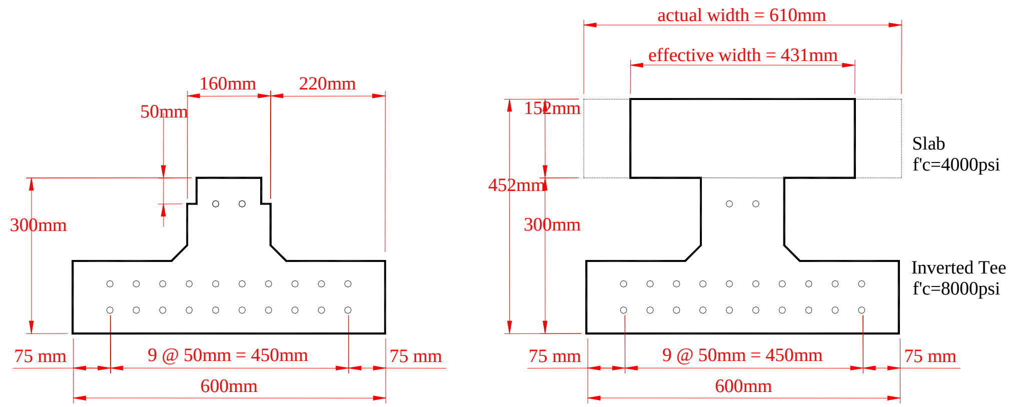
Figure A2: Comparison of New and Existing IT Shapes

	Area mm^2		
Section	**Std.**	**Modified**	**New / Old**
IT300	109,100	113,700	104.2%
IT400	125,100	129,700	103.7%
IT500	141,100	145,700	103.3%
IT600	157,100	161,700	102.9%
IT700	173,100	177,700	102.7%
IT800	189,100	193,700	102.4%
IT900	205,100	209,700	102.2%

	Moment of Inertia (I) mm^4 x 10^6		
Section	**Std.**	**Modified**	**New / Old**
IT300	590	609	103.3%
IT400	1397	1402	100.4%
IT500	2755	2754	99.9%
IT600	4768	4772	100.1%
IT700	7528	7552	100.3%
IT800	11126	11185	100.5%
IT900	15649	15761	100.7%

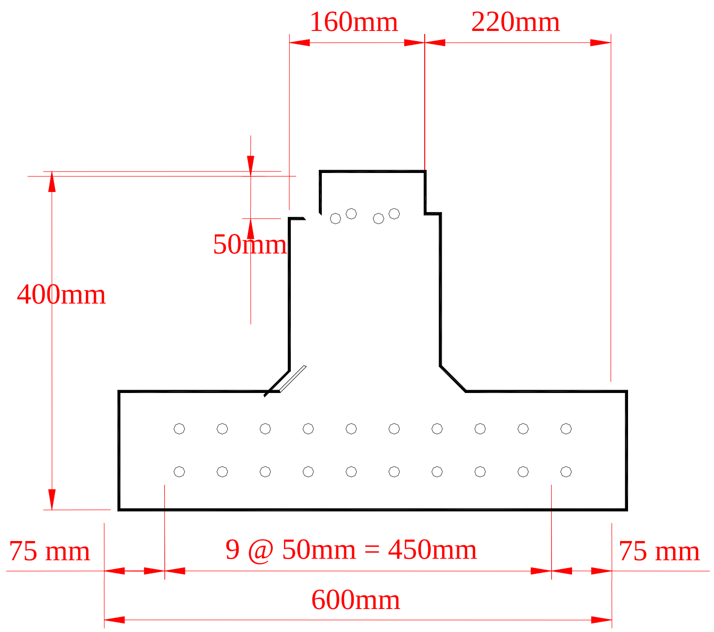
Figure A3: Comparison of Section Properties for both the Existing (Standard) Section and the New (Modified) Section

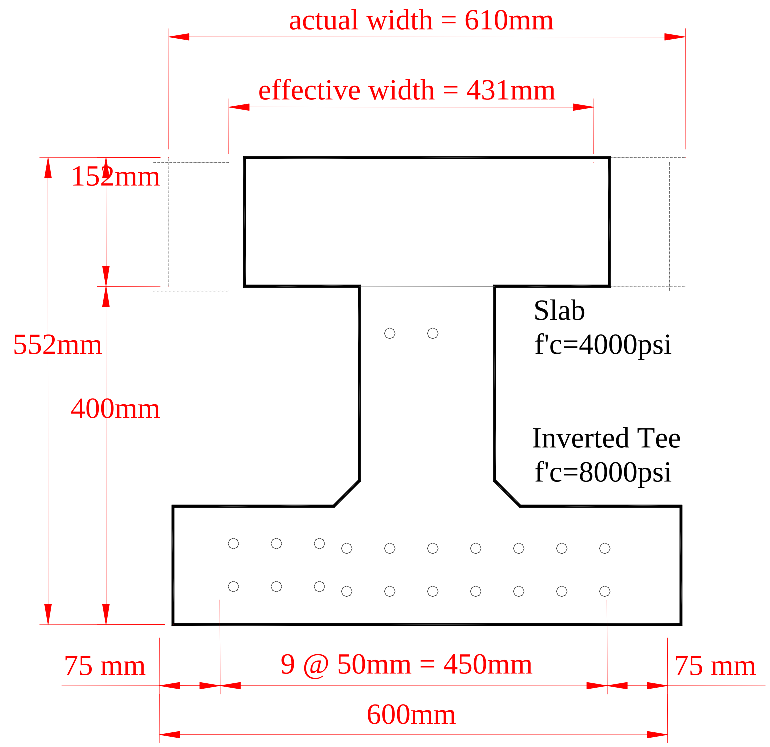
IT300 Standard



	Untopped Section Properties				Topped Section Properties		
A	109056 mm^2	=	169.0 in^2	**A**	176012 mm^2	=	272.8 in^2
$\mathbf{y_t}$	197 mm	=	7.75 in	$\mathbf{y_t}$	246 mm	=	9.68 in
$\mathbf{y_b}$	103 mm	=	4.06 in	$\mathbf{y_b}$	206 mm	=	8.11 in
I	589834718 mm^4	=	1417 in^4	**I**	3772789056 mm^4	=	9064 in^4
$\mathbf{S_t}$	2995265 mm^3	=	183 in^3	$\mathbf{S_t}$	15343997 mm^3	=	936 in^3
$\mathbf{S_b}$	5722237 mm^3	=	349 in^3	$\mathbf{S_b}$	18303889 mm^3	=	1117 in^3

IT400 Standard





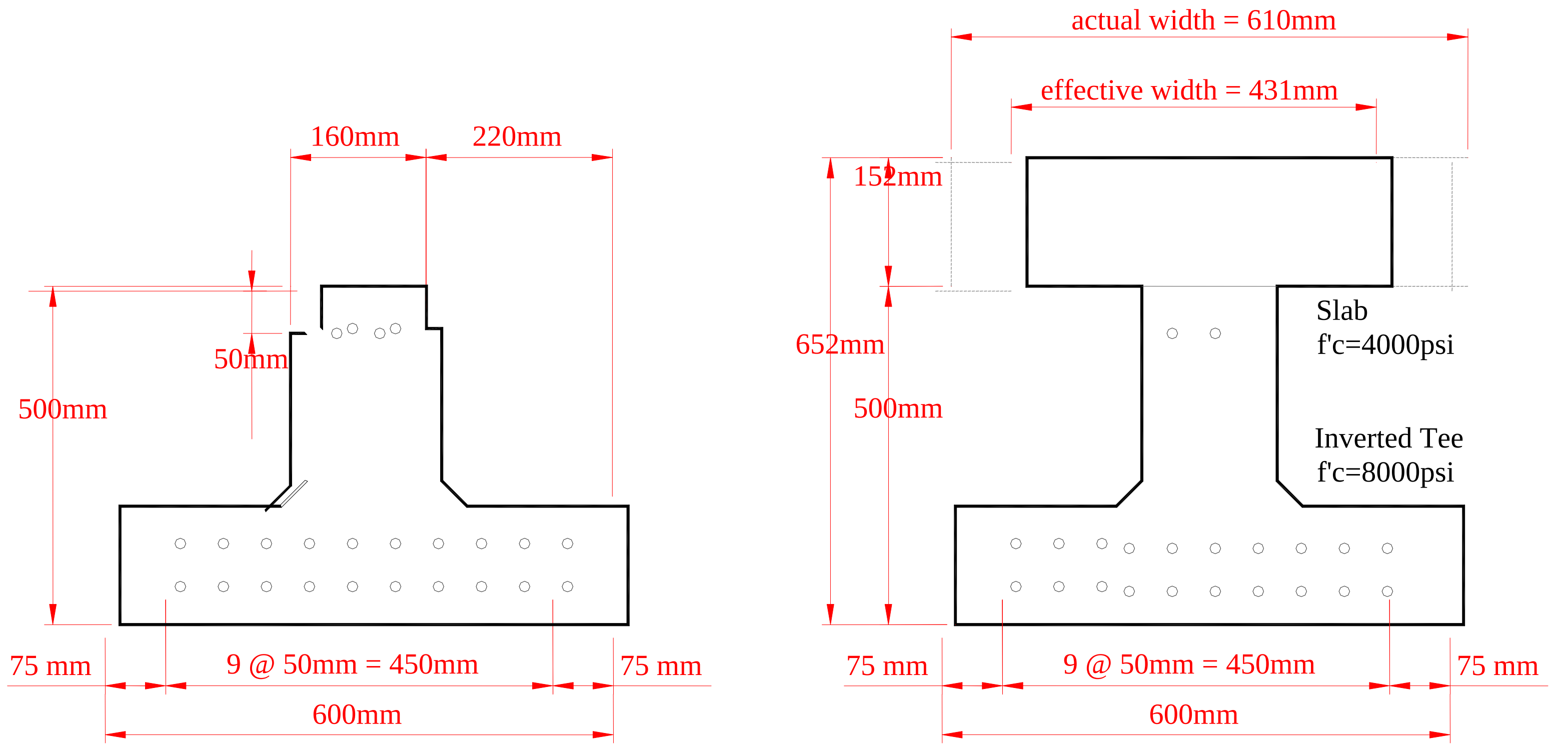
Untopped Section Properties

A	125056 mm^2	=	193.8 in^2
y_t	266 mm	=	10.49 in
y_b	134 mm	=	5.26 in
I	1397019478 mm^4	=	3356 in^4
S_t	5242392 mm^3	=	320 in^3
S_b	10463400 mm^3	=	639 in^3

Topped Section Properties

A	192012 mm^2	=	297.6 in^2
y_t	300 mm	=	11.80 in
y_b	252 mm	=	9.93 in
I	6590102276 mm^4	=	15833 in^4
S_t	21983681 mm^3	=	1342 in^3
S_b	26127609 mm^3	=	1594 in^3

IT500 Standard



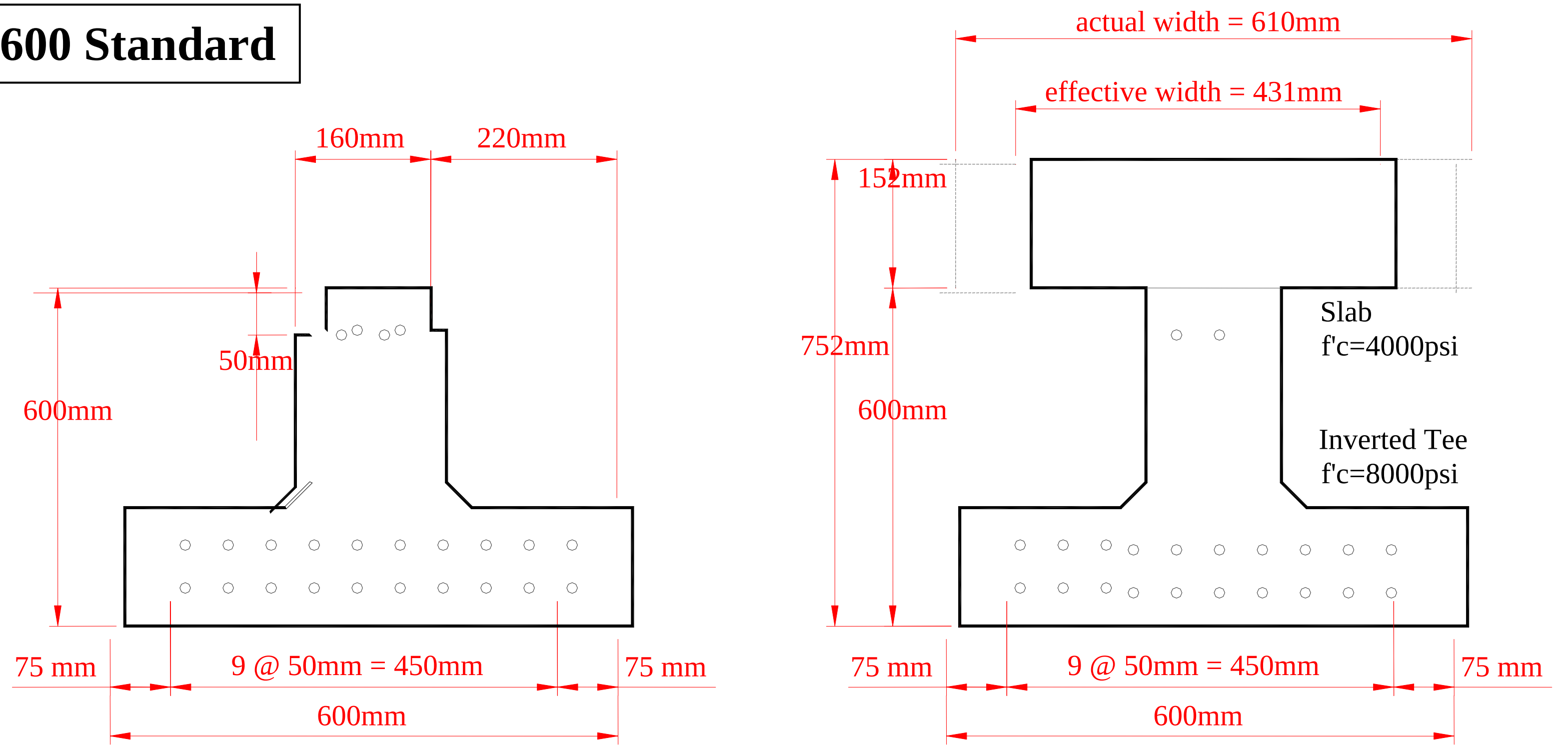
Untopped Section Properties

A	141056 mm^2	=	218.6 in^2
y_t	332 mm	=	13.06 in
y_b	168 mm	=	6.63 in
I	2755480535 mm^4	=	6620 in^4
S_t	8309404 mm^3	=	507 in^3
S_b	16363672 mm^3	=	999 in^3

Topped Section Properties

A	208012 mm^2	=	322.4 in^2
y_t	353 mm	=	13.90 in
y_b	299 mm	=	11.77 in
I	10362553286 mm^4	=	24896 in^4
S_t	29350209 mm^3	=	1791 in^3
S_b	34664991 mm^3	=	2115 in^3

IT600 Standard



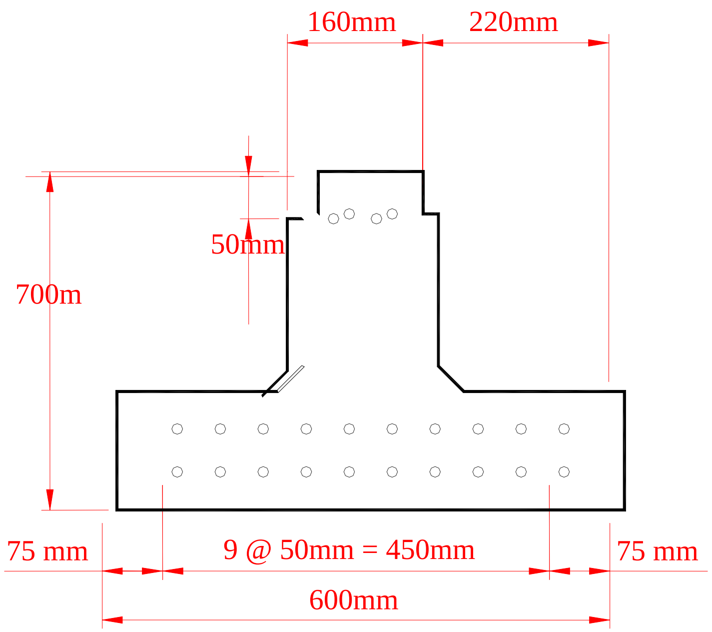
Untopped Section Properties

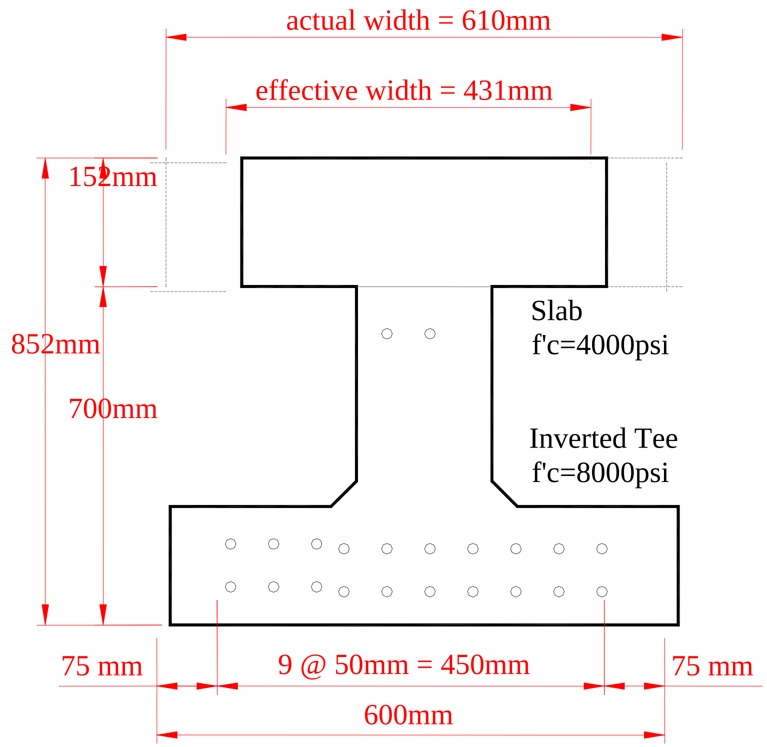
A	157056 mm^2	=	243.4 in^2
$\mathbf{y_t}$	394 mm	=	15.50 in
$\mathbf{y_b}$	206 mm	=	8.12 in
I	4767835160 mm^4	=	11455 in^4
$\mathbf{S_t}$	12111772 mm^3	=	739 in^3
$\mathbf{S_b}$	23105909 mm^3	=	1410 in^3

Topped Section Properties

A	224012 mm^2	=	347.2 in^2
$\mathbf{y_t}$	406 mm	=	15.98 in
$\mathbf{y_b}$	346 mm	=	13.63 in
I	15171196790 mm^4	=	36449 in^4
$\mathbf{S_t}$	37377738 mm^3	=	2281 in^3
$\mathbf{S_b}$	43833274 mm^3	=	2675 in^3

IT700 Standard





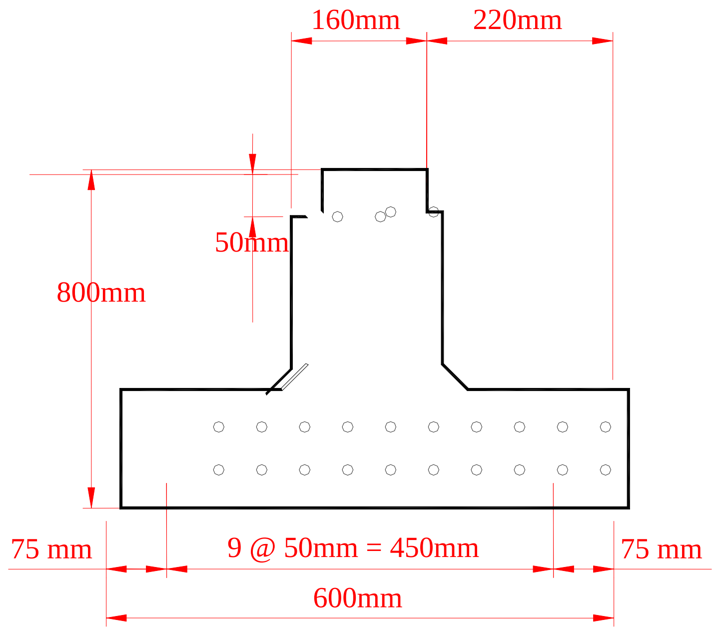
Untopped Section Properties

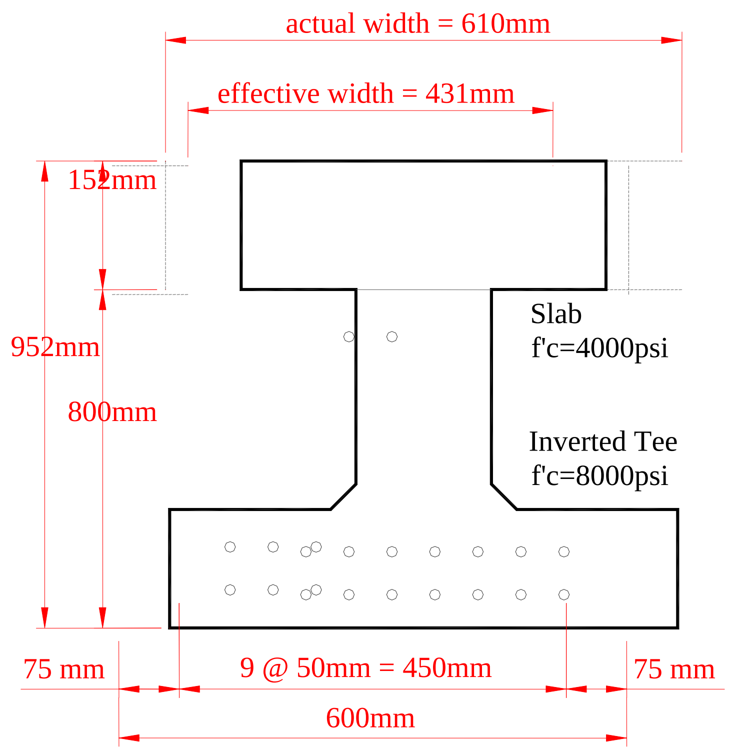
A	173056 mm^2	=	268.2 in^2
y_t	453 mm	=	17.85 in
y_b	247 mm	=	9.71 in
I	7528336245 mm^4	=	18087 in^4
S_t	16601649 mm^3	=	1013 in^3
S_b	30537101 mm^3	=	1863 in^3

Topped Section Properties

A	240012 mm^2	=	372.0 in^2
y_t	458 mm	=	18.04 in
y_b	394 mm	=	15.50 in
I	21096806249 mm^4	=	50685 in^4
S_t	46029223 mm^3	=	2809 in^3
S_b	53590766 mm^3	=	3270 in^3

IT800 Standard





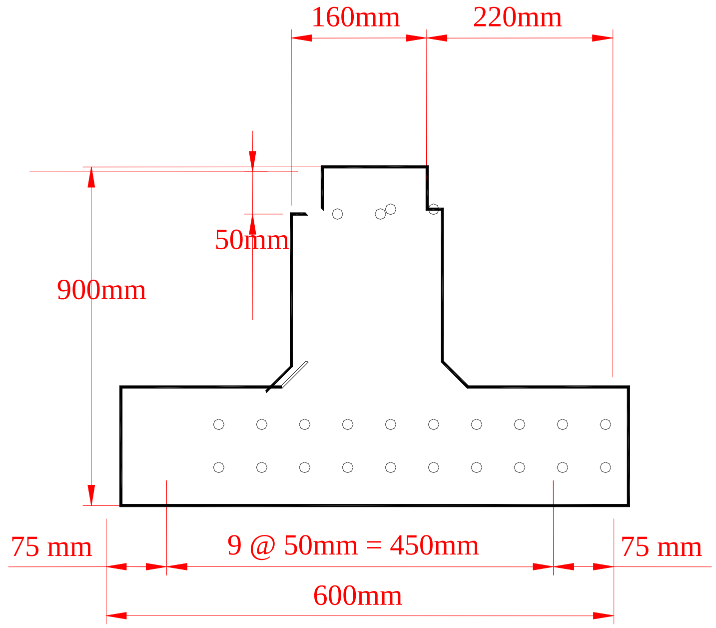
Untopped Section Properties

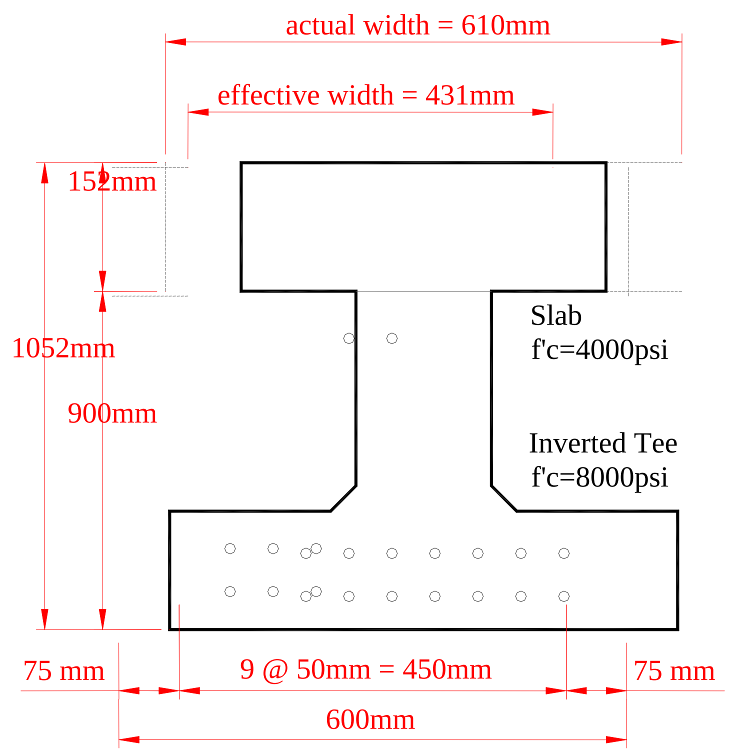
A	189056 mm^2	=	293.0 in^2
y_t	512 mm	=	20.14 in
y_b	288 mm	=	11.35 in
I	11126411737 mm^4	=	26731 in^4
S_t	21747249 mm^3	=	1327 in^3
S_b	38582986 mm^3	=	2354 in^3

Topped Section Properties

A	256012 mm^2	=	396.8 in^2
y_t	510 mm	=	20.10 in
y_b	442 mm	=	17.38 in
I	28219961770 mm^4	=	67799 in^4
S_t	55281692 mm^3	=	3373 in^3
S_b	63914859 mm^3	=	3900 in^3

IT900 Standard





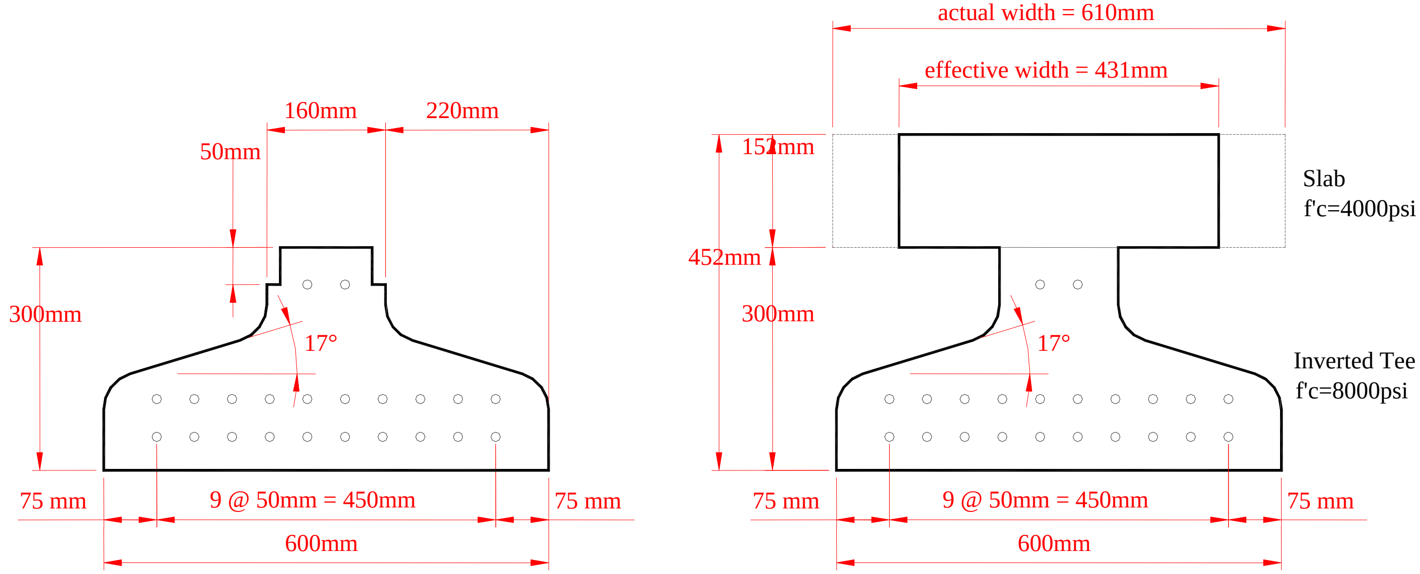
Untopped Section Properties

A	205056 mm^2	=	317.8 in^2
y_t	569 mm	=	22.38 in
y_b	331 mm	=	13.05 in
I	15648547029 mm^4	=	37596 in^4
S_t	27525738 mm^3	=	1680 in^3
S_b	47206123 mm^3	=	2881 in^3

Topped Section Properties

A	272012 mm^2	=	421.6 in^2
y_t	562 mm	=	22.14 in
y_b	490 mm	=	19.28 in
I	36621106969 mm^4	=	87983 in^4
S_t	65119868 mm^3	=	3974 in^3
S_b	74792601 mm^3	=	4564 in^3

IT300 Modified



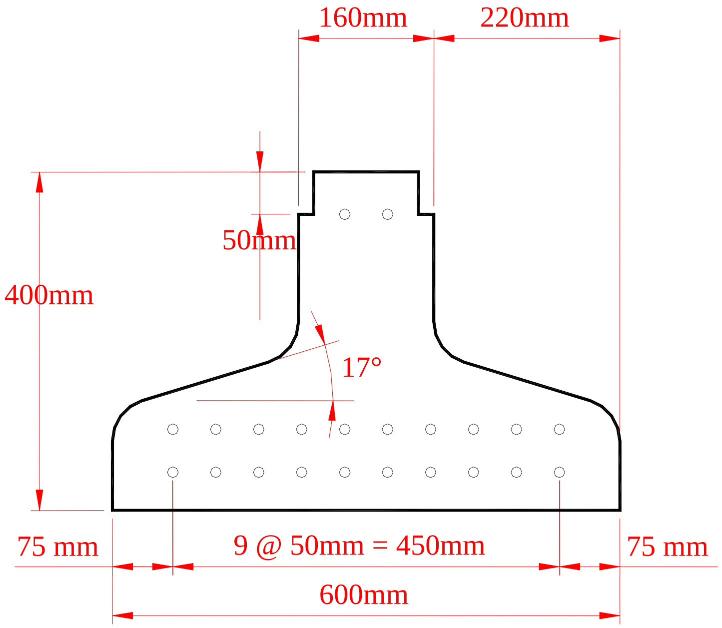
Untopped Section Properties

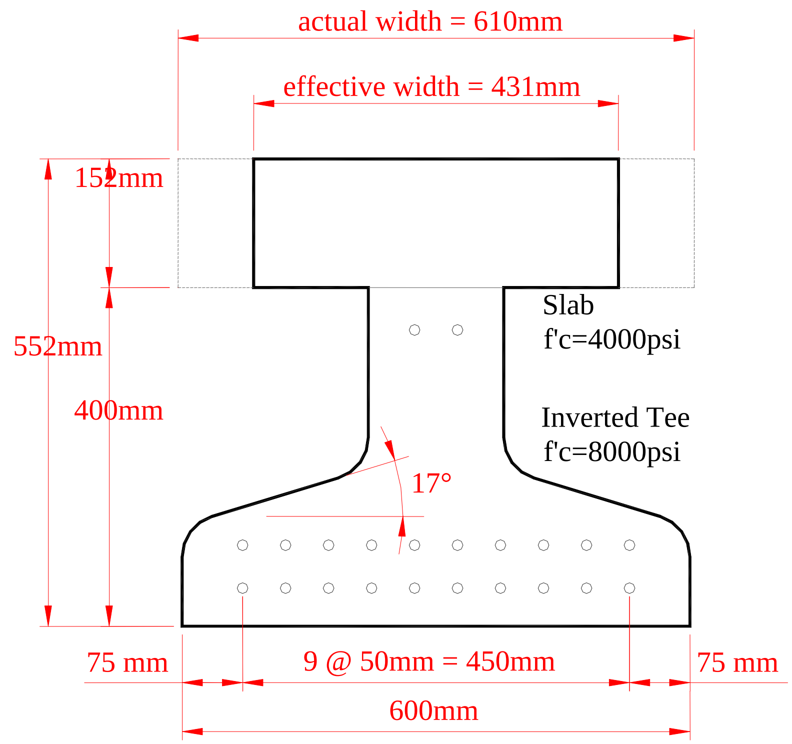
A	113656 mm^2	=	176.2 in^2
y_t	194 mm	=	7.64 in
y_b	106 mm	=	4.17 in
I	609370275 mm^4	=	1464 in^4
S_t	3139059 mm^3	=	192 in^3
S_b	5755570 mm^3	=	351 in^3

Topped Section Properties

A	180612 mm^2	=	279.9 in^2
y_t	247 mm	=	9.71 in
y_b	205 mm	=	8.08 in
I	3776401722 mm^4	=	9073 in^4
S_t	15304902 mm^3	=	934 in^3
S_b	18398549 mm^3	=	1123 in^3

IT400 Modified





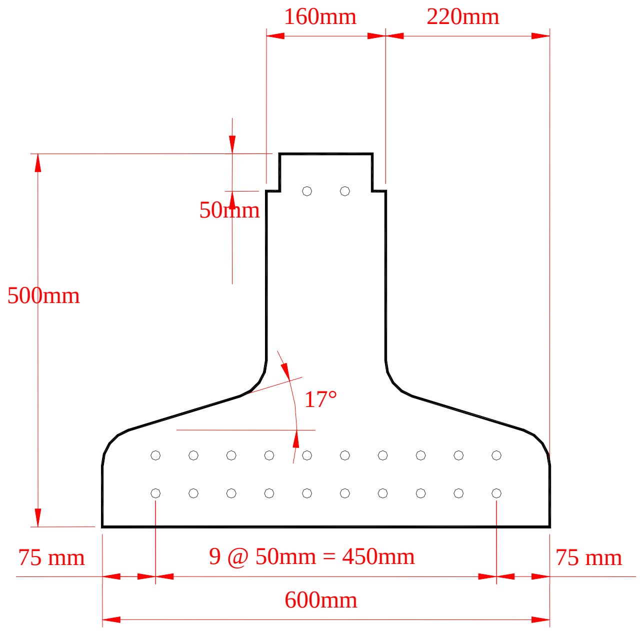
Untopped Section Properties

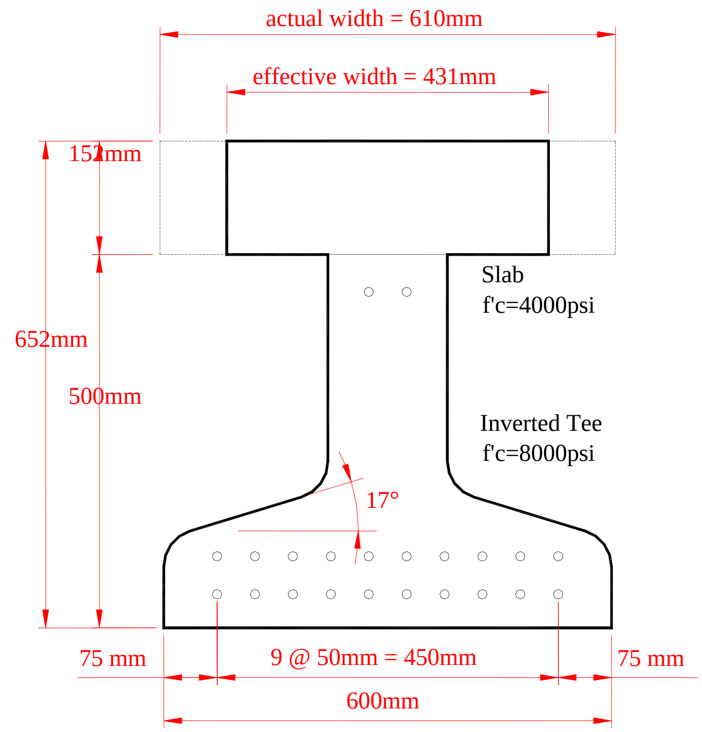
A	129656 mm^2	=	201.0 in^2
y_t	265 mm	=	10.44 in
y_b	135 mm	=	5.31 in
I	1402108477 mm^4	=	3369 in^4
S_t	5288721 mm^3	=	323 in^3
S_b	10394686 mm^3	=	634 in^3

Topped Section Properties

A	196612 mm^2	=	304.7 in^2
y_t	302 mm	=	11.88 in
y_b	250 mm	=	9.86 in
I	6617332021 mm^4	=	15898 in^4
S_t	21937480 mm^3	=	1339 in^3
S_b	26431799 mm^3	=	1613 in^3

IT500 Modified





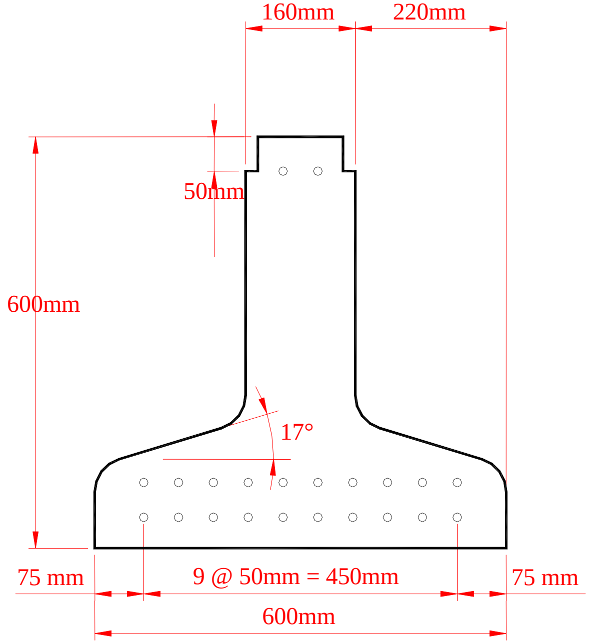
Untopped Section Properties

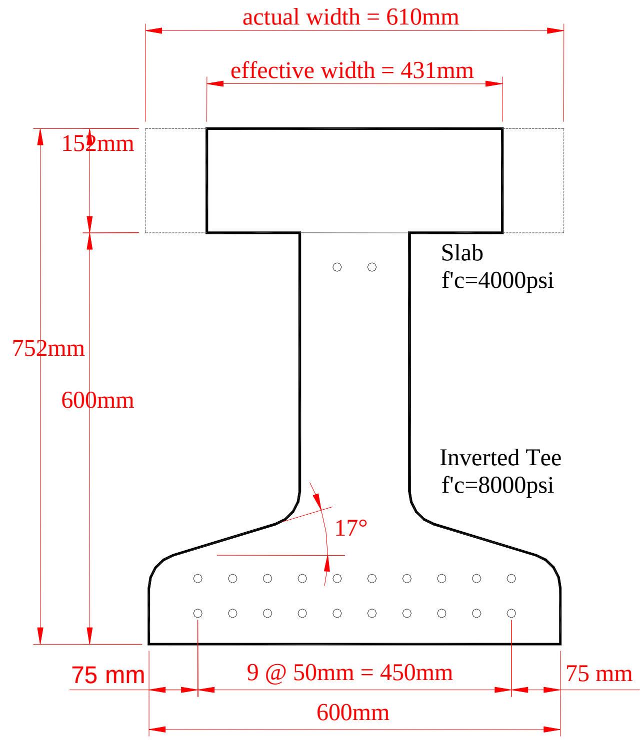
A	145656 mm^2	=	225.8 in^2
y_t	331 mm	=	13.05 in
y_b	169 mm	=	6.63 in
I	2753997145 mm^4	=	6620 in^4
S_t	8307938 mm^3	=	507 in^3
S_b	16343212 mm^3	=	997 in^3

Topped Section Properties

A	212612 mm^2	=	329.5 in^2
y_t	356 mm	=	14.01 in
y_b	296 mm	=	11.66 in
I	10433300550 mm^4	=	25066 in^4
S_t	29322845 mm^3	=	1789 in^3
S_b	35224779 mm^3	=	2150 in^3

IT600 Modified





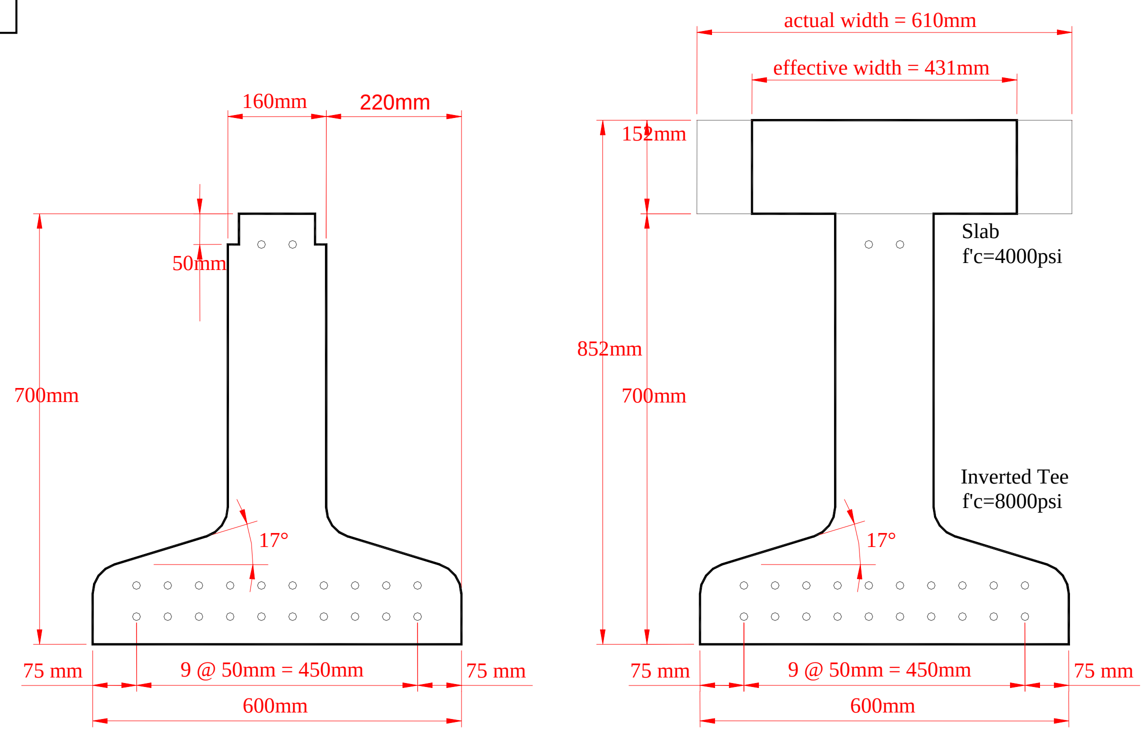
Untopped Section Properties

A	161656 mm^2	=	250.6 in^2
y_t	395 mm	=	15.54 in
y_b	205 mm	=	8.09 in
I	4771501227 mm^4	=	11464 in^4
S_t	12091232 mm^3	=	738 in^3
S_b	23233107 mm^3	=	1418 in^3

Topped Section Properties

A	228612 mm^2	=	354.3 in^2
y_t	409 mm	=	16.12 in
y_b	343 mm	=	13.49 in
I	15305991583 mm^4	=	36773 in^4
S_t	37387485 mm^3	=	2282 in^3
S_b	44674431 mm^3	=	2726 in^3

IT700 Modified



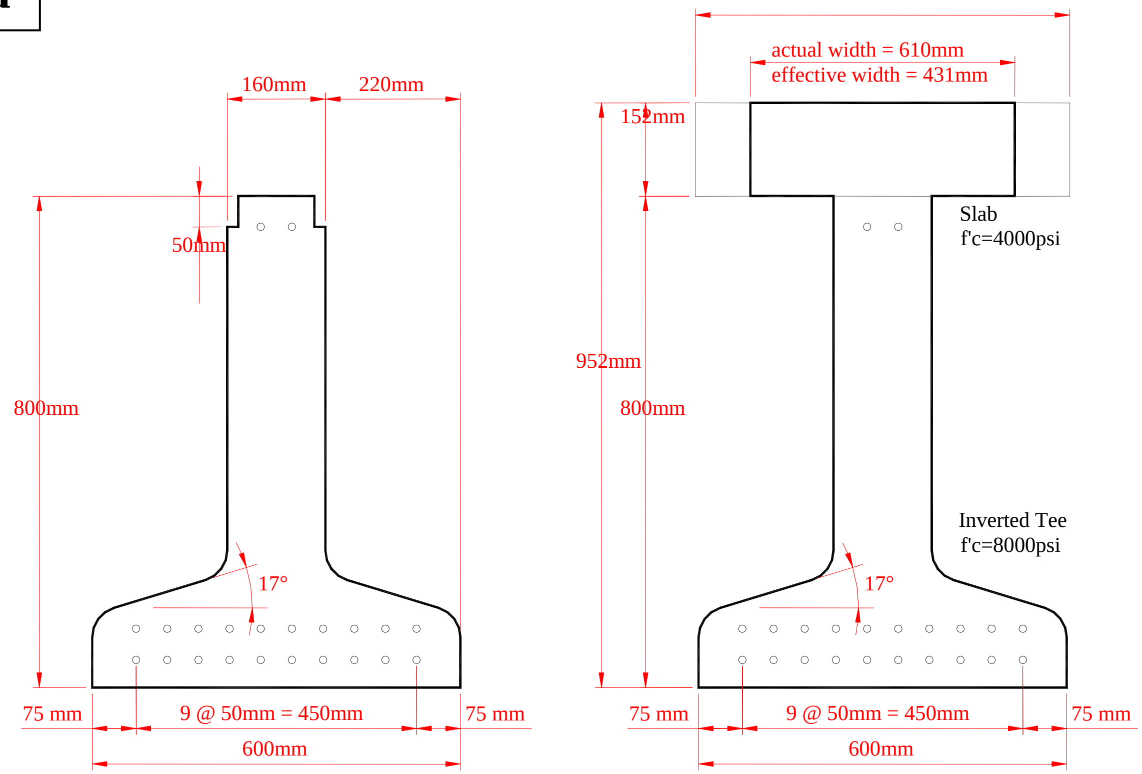
Untopped Section Properties

A	177656 mm^2	=	275.4 in^2
y_t	455 mm	=	17.93 in
y_b	245 mm	=	9.63 in
I	7551551758 mm^4	=	18143 in^4
S_t	16582456 mm^3	=	1012 in^3
S_b	30872314 mm^3	=	1884 in^3

Topped Section Properties

A	244612 mm^2	=	379.1 in^2
y_t	462 mm	=	18.21 in
y_b	390 mm	=	15.33 in
I	21316648720 mm^4	=	51213 in^4
S_t	46090058 mm^3	=	2813 in^3
S_b	54728229 mm^3	=	3340 in^3

IT800 Modified



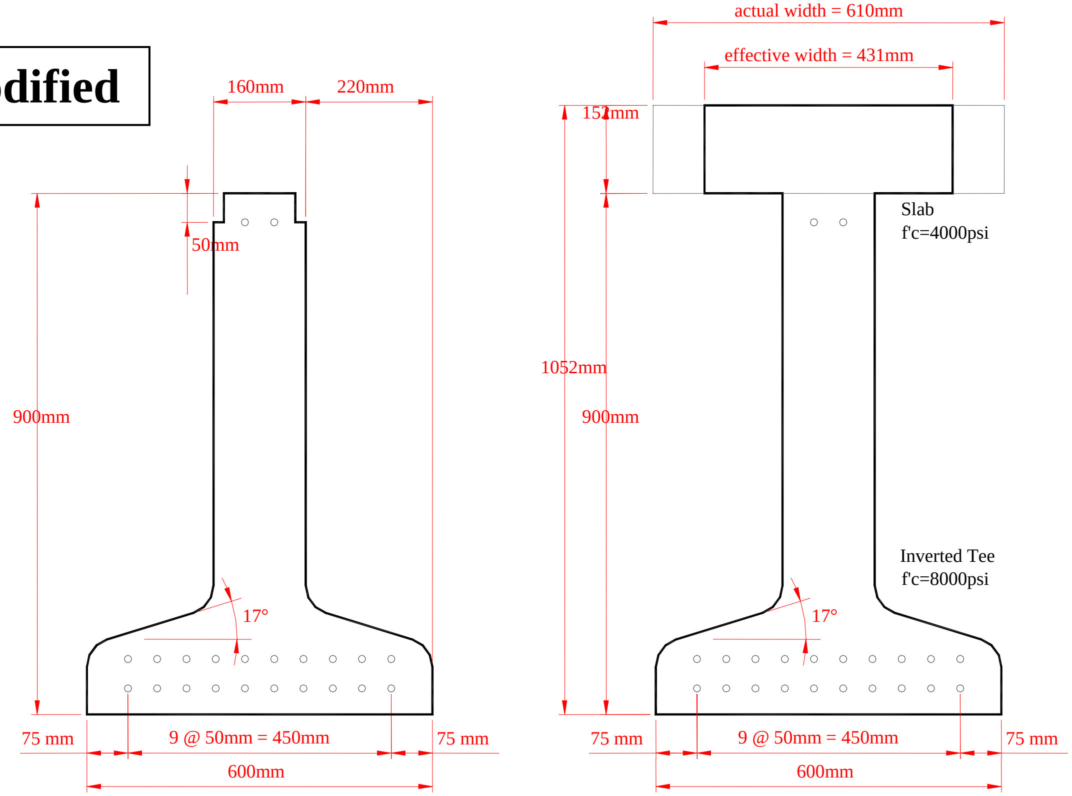
Untopped Section Properties

A	193656 mm^2	=	300.2 in^2
y_t	514 mm	=	20.25 in
y_b	286 mm	=	11.24 in
I	11185484358 mm^4	=	26873 in^4
S_t	21745411 mm^3	=	1327 in^3
S_b	39162623 mm^3	=	2390 in^3

Topped Section Properties

A	260612 mm^2	=	403.9 in^2
y_t	515 mm	=	20.28 in
y_b	437 mm	=	17.20 in
I	28546210166 mm^4	=	68583 in^4
S_t	55404826 mm^3	=	3381 in^3
S_b	65357484 mm^3	=	3988 in^3

IT900 Modified



Untopped Section Properties

A	209656 mm^2	=	325.0 in^2
y_t	572 mm	=	22.52 in
y_b	328 mm	=	12.91 in
I	15761174309 mm^4	=	37866 in^4
S_t	27554442 mm^3	=	1681 in^3
S_b	48052538 mm^3	=	2932 in^3

Topped Section Properties

A	276612 mm^2	=	428.7 in^2
y_t	568 mm	=	22.35 in
y_b	484 mm	=	19.07 in
I	37075397050 mm^4	=	89074 in^4
S_t	65314568 mm^3	=	3986 in^3
S_b	76545696 mm^3	=	4671 in^3

Appendix B

Sample Calculations for the Design of a 3-Span IT Bridge

IT 500 60 ft 3-Spans
"Checking with Conspan"

Input Data:

Geometry

Bridge Layout

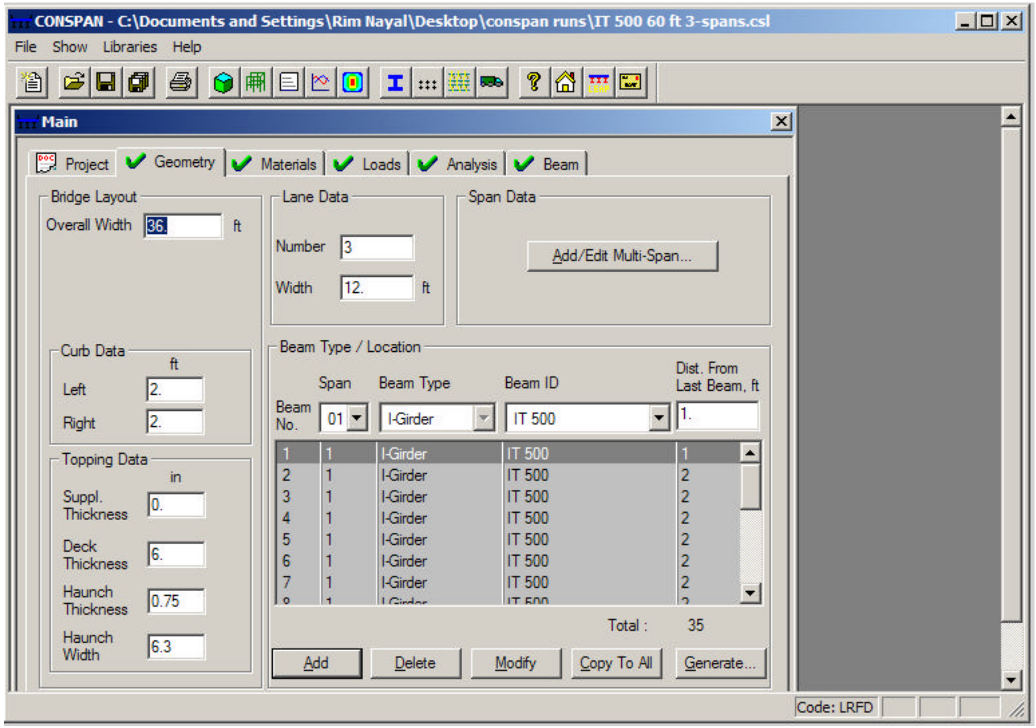
Overall Bridge Width $Bridgewid = 36ft$

Left Curb $LCurb = 2ft$

Right Curb $RCurb = 2ft$

Num of Lanes $NL = 3$

Lane Width $LW = 12ft$



Topping Data

Supplementry Thickness $ST = 0in$

Deck Thickness $DT = 6in$

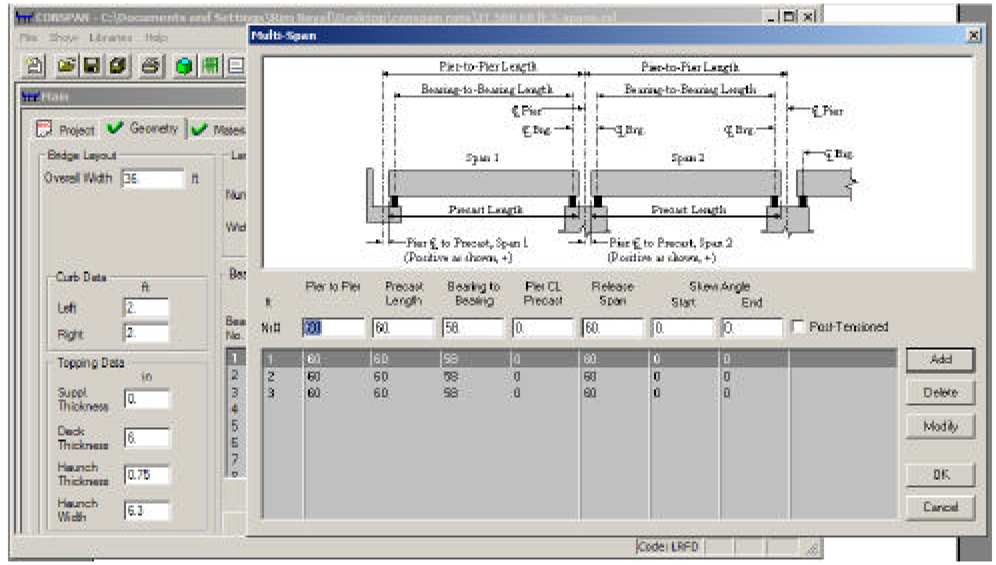
Haunch Thickness $HT = 0.75in$

Haunch Width $HW = 6.3in$

Span Data

Pier To Pier	$PTP = 60ft$
Precast Length	$PL = 60ft$
Bearing To Bearing	$BTB = 58ft$
Pier CL to Precast	$PTPR = 0ft$
Release Span	$RS = 60ft$

$$pcf = \frac{lb}{ft^3}$$



Concrete

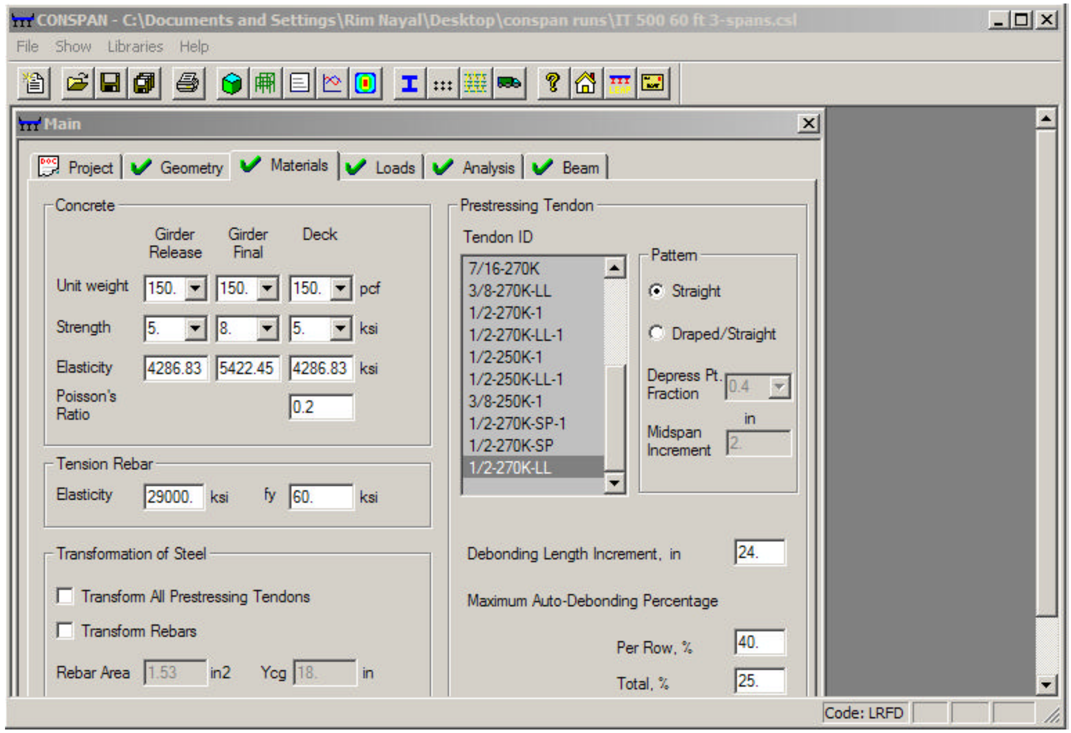
Formulations Used

$$E_c(fc, Wl) = 0.001 \times 33Wl^{1.5}\sqrt{\left(\frac{fc}{144}\right)}$$

$$\beta1(fc) = \text{if}\left[\frac{fc}{ksi} < 4, 0.85, \text{if}\left[\left[0.85 - \left(\frac{fc}{ksi} - 4\right)0.05\right] < 0.65, 0.65, 0.85 - \left(\frac{fc}{ksi} - 4\right)0.05\right]\right]$$

	Girder Release	Girder Final	Deck
Unit Weight	$Wl_{fgr} = 150pcf$	$Wl_{fgf} = 150pcf$	$Wl_{fd} = 150pcf$
Strength	$fc_{gr} = 5ksi$	$fc_{gf} = 8ksi$	$fc_d = 5ksi$
Elasticity	$E_c(fc_{gr}, Wl_{fgr}) = 4.287 \times 10^3\,ksi$	$E_c(fc_{gf}, Wl_{fgf}) = 5.422 \times 10^3\,ksi$	$E_c(fc_d, Wl_{fd}) = 4.287 \times 10^3\,ksi$
$\beta1$	$\beta1(fc_{gr}) = 0.8$	$\beta1(fc_{gf}) = 0.65$	$\beta1(fc_d) = 0.8$

Poisson Ratio $\nu = 0.2$ $\quad n = \dfrac{E_c(fc_d, Wl_{fd})}{E_c(fc_{gf}, Wl_{fgf})}$ $\quad n = 0.791$

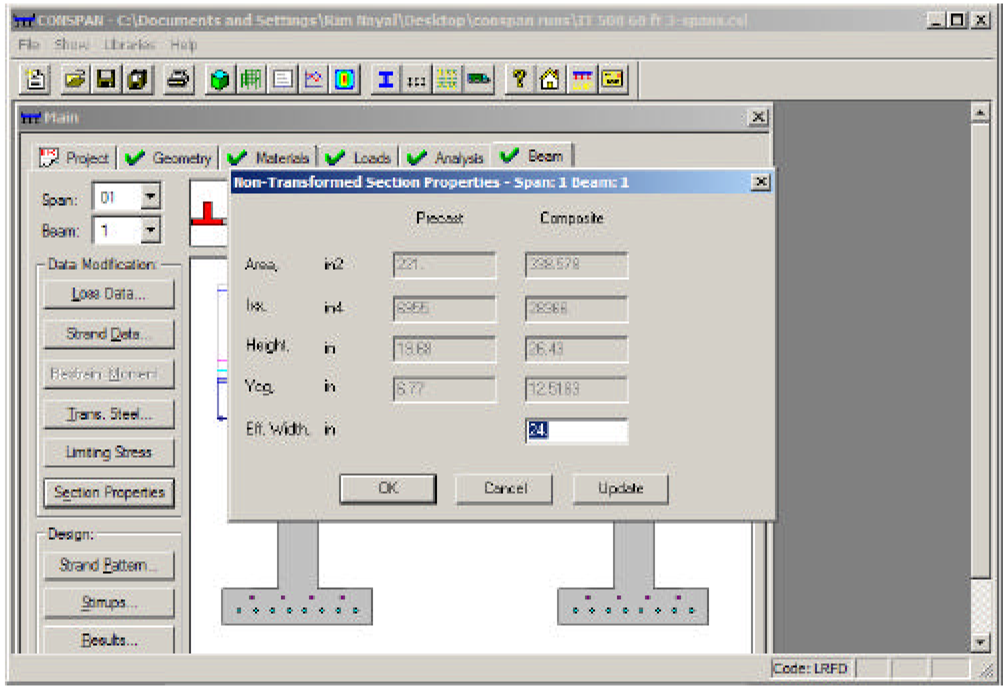


Section Properties

Name :IT500

Hight	h = 19.68in
Bottom flange width	bw = 24in
Stem width	ww = 6.3in
Flange Height	hf = 6in

Non Composite Section Properties:



$$A = 221\,\text{in}^2$$

$$I = 6955\,\text{in}^4$$

$$y_b = 6.77\,\text{in}$$

$$y_t = h - y_b \qquad y_t = 12.91\,\text{in}$$

$$s_b = \frac{I}{y_b} \qquad s_b = 1.027 \times 10^3\,\text{in}^3$$

$$s_t = \frac{I}{y_t} \qquad s_t = 538.73\,\text{in}^3$$

Composite Section Properties:

$$h_c = h + DT + HT \qquad h_c = 26.43\,\text{in}$$

$$A_c = A + (DT\,bw\,n) + (HT\,ww\,n) \qquad A_c = 338.577\,\text{in}^2$$

$$y_{bc} = \frac{A\,y_b + DT\,bw\,n\left(h_c - \frac{DT}{2}\right) + HT\,ww\,n\left(h + \frac{HT}{2}\right)}{A_c} \qquad y_{bc} = 12.518\,\text{in}$$

$$y_{tc} = h + DT + HT - y_{bc} \qquad y_{tc} = 13.912\,\text{in}$$

$$I_c = I + A\left(y_{bc} - y_b\right)^2 + \frac{bw\,n\,DT^3}{12} + bw\,n\,DT\left(h_c - \frac{DT}{2} - y_{bc}\right)^2 + \frac{ww\,n\,HT^3}{12} + ww\,n\,HT\left(h + \frac{HT}{2} - y_{bc}\right)^2$$

$$I_c = 2.837 \times 10^4\,\text{in}^4$$

$$s_{bc} = \frac{I_c}{y_{bc}} \qquad s_{bc} = 2.266 \times 10^3\,in^3$$

$$s_{tc} = \frac{I_c}{y_{tc}} \qquad s_{tc} = 2.039 \times 10^3\,in^3$$

Tendons $\frac{1}{2} - 270K - LL$

Diameter $D = 0.5in$

Tendons area $A_{sp} = 0.153in^2$

$E_p = 28500ksi$

$k = 0.28$

Ultimate Stress $f_{pu} = 270ksi$

Yielding stress $f_{py} = 0.9f_{pu}$ $f_{py} = 243\,ksi$

$Jacking\_ratio = 0.75$ Jacking stress $f_j = Jacking\_ratio\,f_{pu}$ $f_j = 202.5\,ksi$

Tendons Positions

Eccentricity	Num. of Strands
$e_{c1} = 17.68in$	$n_1 = 2$
$e_{c2} = 4in$	$n_2 = 4$
$e_{c3} = 2in$	$n_3 = 8$

$$y_{bs} = \frac{e_{c1}n_1 + e_{c2}n_2 + e_{c3}n_3}{n_1 + n_2 + n_3} \qquad y_{bs} = 4.811\,in$$

$$\Sigma A_{sp} = (n_1 + n_2 + n_3)A_{sp} \qquad \Sigma A_{sp} = 2.142\,in^2$$

$$p_j = \Sigma A_{sp} f_j \qquad p_j = 433.755\,kips$$

$$e_{cc} = y_b - y_{bs} \qquad e_{cc} = 1.959\,in$$

Loss Data

Release Time $RT = 0.75\text{day}$

Relative Humidity $RH = 75\ \%$

Loads:

$$SW_{girder} = Wl_{fgf} A$$

$$SW_{HD} = Wl_{fd}(DTbw + HTww)$$

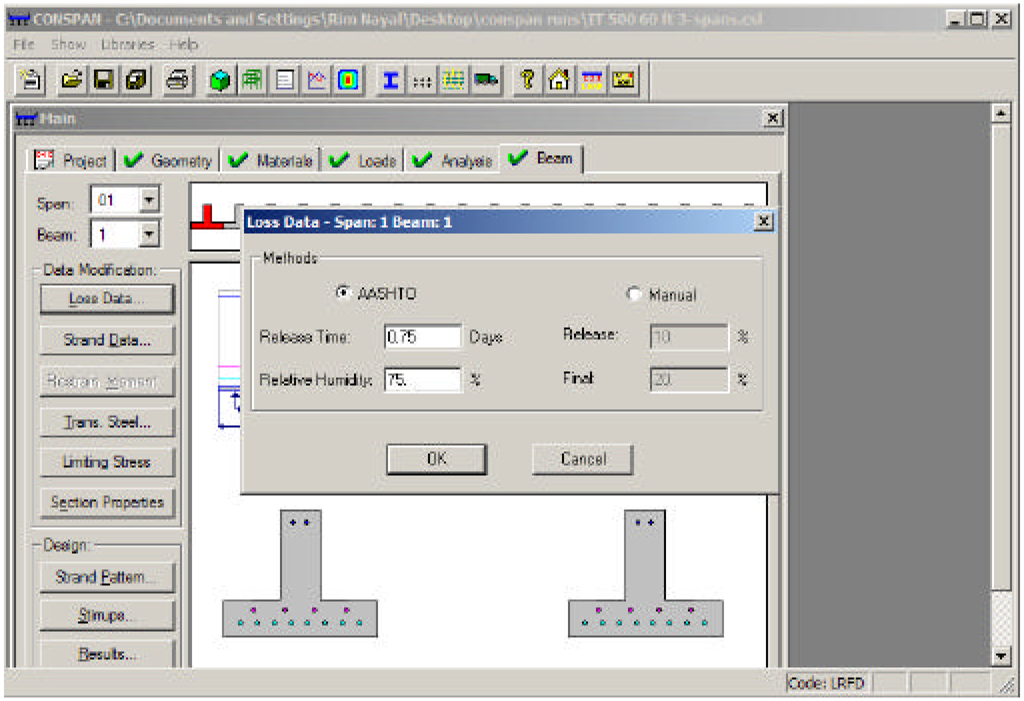
$$\text{maintainance} = 0.02\text{ksf}$$

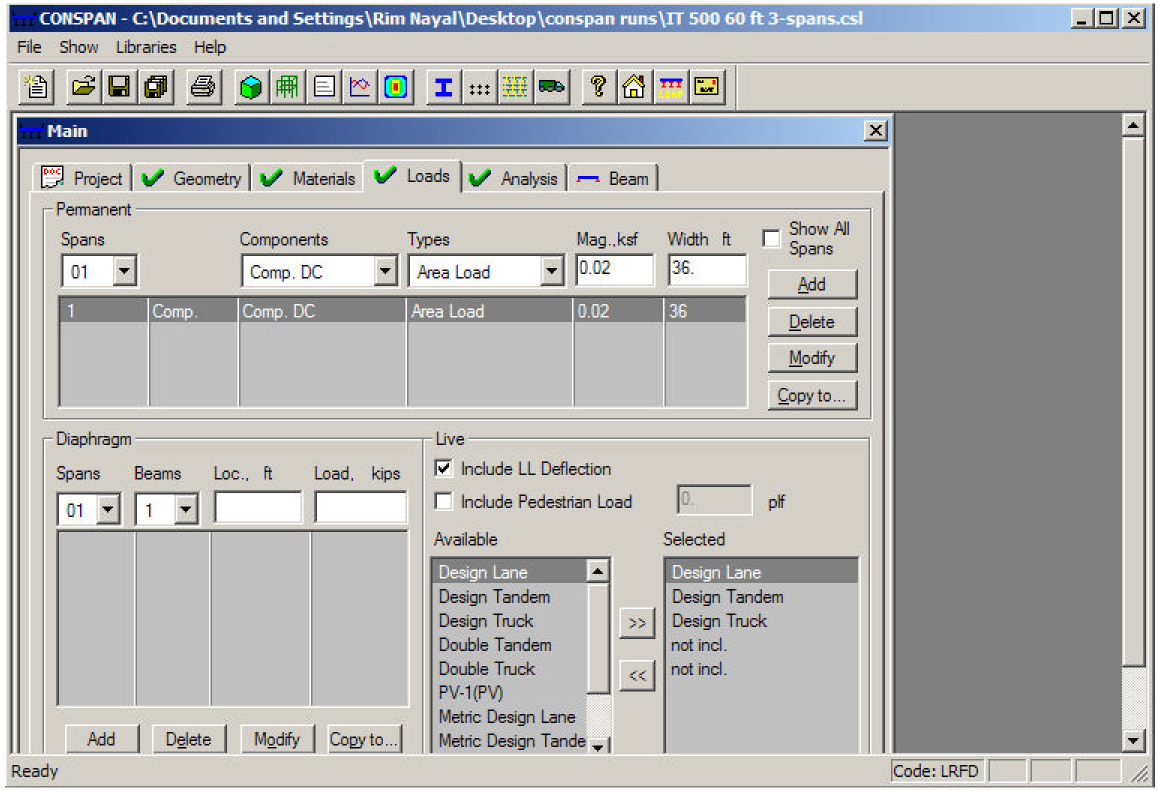
$$SW_{girder} = 230.208\,\frac{\text{lb}}{\text{ft}}$$

$$SW_{HD} = 154.922\,\frac{\text{lb}}{\text{ft}}$$

$$ml = \text{maintainance}\,\text{Bridgewid}$$

$$ml = 720\,\frac{\text{lb}}{\text{ft}}$$





values of shear forces and bending moments are for a simply supported interior beam under self weight weight of slab and haunch, in this case the design span is used.

the shear forces and bending moments due to other loadings are calculated based on the continious span length.

Distribution factors:

Positive Moment

$PM_{DF} = 0.2$

Negative Moment

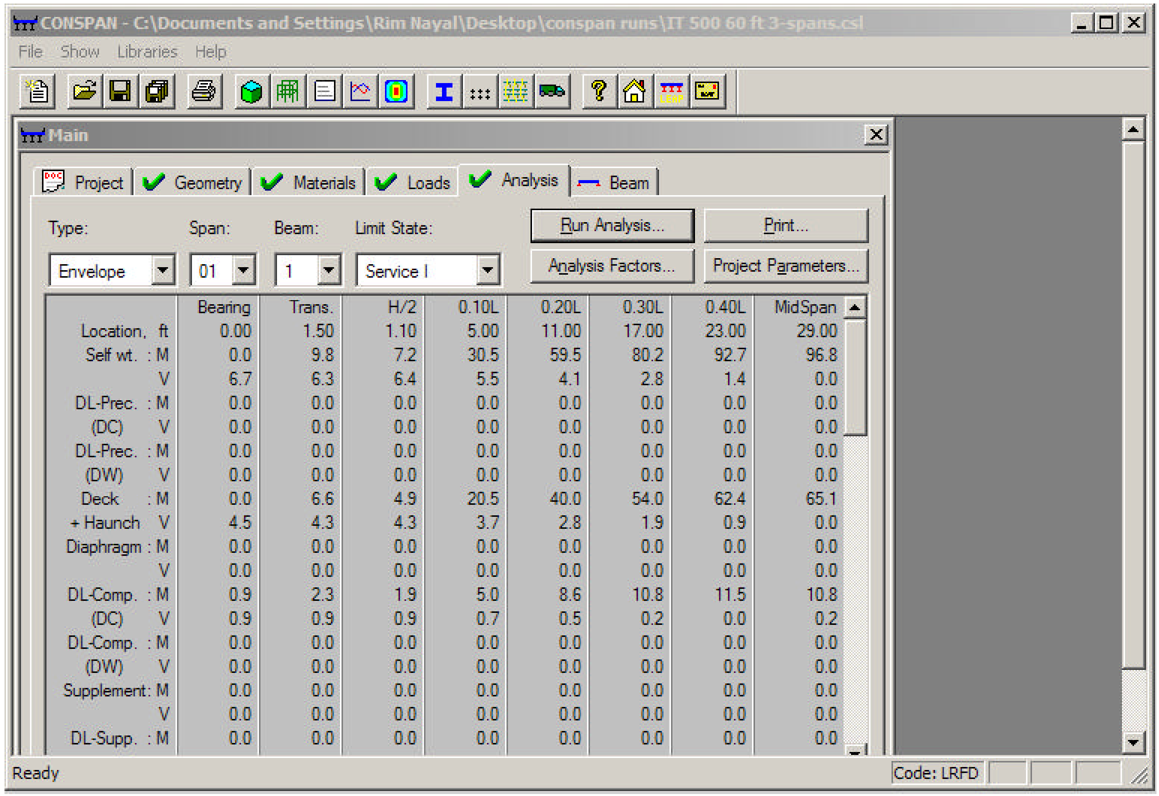
$NM_{DF} = 0.2$

Shear

$V_{DF} = 0.42$

Impact Factor:

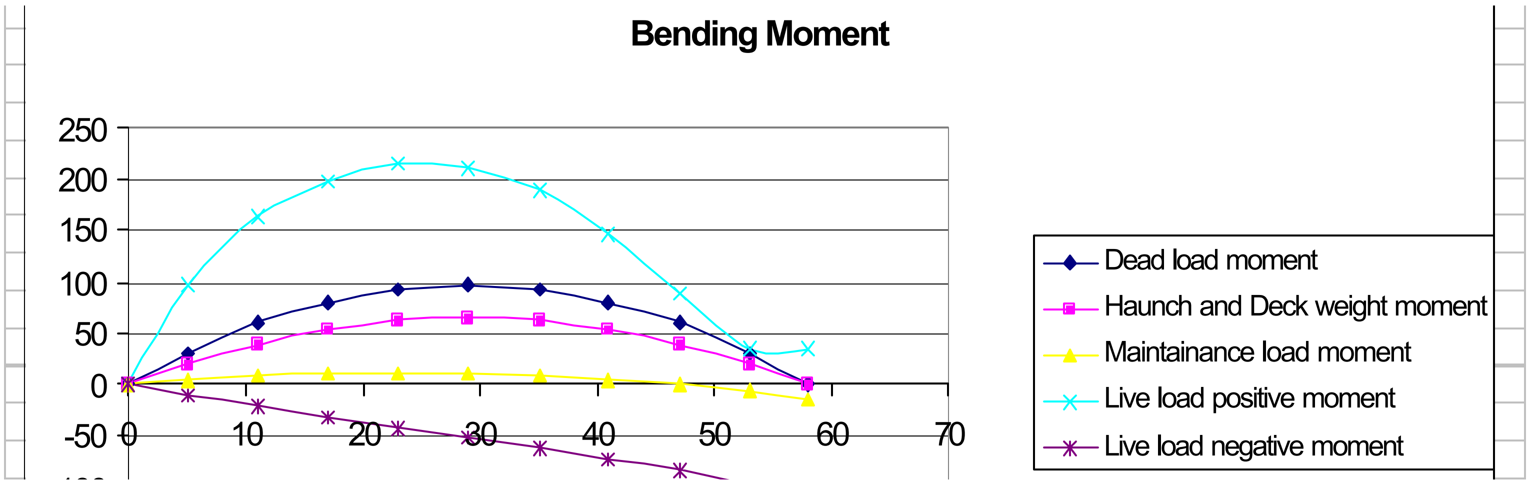
$IM = 0.33$



	Bearing	Trans.	H/2	0.10L	0.20L	0.30L	0.40L	MidSpan
Location, ft	0.00	1.50	1.10	5.00	11.00	17.00	23.00	29.00
Self wt. : M	0.0	9.8	7.2	30.5	59.5	80.2	92.7	96.8
V	6.7	6.3	6.4	5.5	4.1	2.8	1.4	0.0
DL-Prec. : M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
(DC) V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. : M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
(DW) V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck : M	0.0	6.6	4.9	20.5	40.0	54.0	62.4	65.1
+ Haunch V	4.5	4.3	4.3	3.7	2.8	1.9	0.9	0.0
Diaphragm : M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Comp. : M	0.9	2.3	1.9	5.0	8.6	10.8	11.5	10.8
(DC) V	0.9	0.9	0.9	0.7	0.5	0.2	0.0	0.2
DL-Comp. : M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
(DW) V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Supplement: M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Supp. : M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Analysis Envelop:

Span Data	60	58																		
S.W.	230	154.92					Fy	Kips												
DF	0.2	0.2	0.42				Mz	K.ft												
			Dead Load		Haunch+Deck		Composite loa		ane*DF*Impact Facto			andem*DF*Impact Facto			ruck*DF*Impact Facto			LL*DF*Impact Factor		
		ation	Fy	Mz	Fy	Mz	Fy	Mz	Fy	Mz+	Mz-	Fy	Mz+	Mz-	Fy	Mz+	Mz-	Fy	Mz+	Mz-
1	1	0	6.68	0	4.49	0	-0.96	0	7.26	0	0	26.75	0	0	32.48	0	0	39.73	0	0
	2	5	5.53	30.5	3.72	20.53	-0.72	5.04	5.64	18.4	-2.3	22.65	66.38	-6.356	26.8	78.86	-8.2	32.45	97.29	-10.46
	3	11	4.14	59.51	2.79	40.05	-0.48	8.64	4.03	32.3	-4.61	19.21	113	-12.71	22.12	130.8	-16	26.15	163	-20.92
	4	17	2.76	80.23	1.86	53.99	-0.24	10.8	2.42	41.5	-6.91	15.88	140.8	-19.07	17.68	157.7	-24	20.1	199.2	-31.38
	5	23	1.38	92.66	0.93	62.36	0	11.52	0.81	46.1	-9.21	12.7	151.1	-25.42	13.54	169.1	-33	14.35	215.2	-41.85
	6	29	0	96.8	0	65.14	0.24	10.8	-1.9	46.1	-11.5	-15.7	147.5	-31.78	-16.86	164.7	-41	-17.6	210.7	-52.31
	7	35	-1.4	92.66	-0.9	62.36	0.48	8.641	-3.5	41.5	-13.8	-18.7	132	-38.13	-21.37	147.8	-49	-22.2	189.2	-62.77
	8	41	-2.8	80.23	-1.9	53.99	0.72	5.041	-5.1	32.3	-16.1	-21.4	104.8	-44.49	-25.6	113.6	-57	-26.5	145.9	-73.23
	9	47	-4.1	59.51	-2.8	40.05	0.96	0.001	-6.7	18.4	-18.4	-23.7	69.01	-50.84	-29.48	66.77	-65	-30.5	87.44	-83.69
	10	53	-5.5	30.5	-3.7	20.53	1.2	-6.48	-8.3	6.9	-27.6	-25.8	28.41	-57.2	-32.94	23.8	-73	-34.1	35.31	-101.1
	11	58	-6.7	6E-14	-4.5	0	1.44	-14.4	-9.9	7.67	-53.7	-27.4	20.34	-81.45	-35.46	26.44	-106	-37.4	34.11	-159.6
2	1	0	6.68	0	4.49	0	-1.2	-14.4	9.41	7.67	-53.7	27.25	20.34	-81.45	33.96	26.44	-106	43.37	34.11	-159.6
	2	5	5.53	30.5	3.72	20.53	-0.96	-7.92	7.79	3.83	-29.2	24.55	33	-71.27	30.09	18.91	-93	37.88	36.84	-121.8
	3	11	4.14	59.51	2.79	40.05	-0.72	-2.88	6.18	13.8	-23	21.81	72.33	-61.09	25.85	71.84	-79	32.03	86.16	-102.4
	4	17	2.76	80.23	1.86	53.99	-0.48	0.722	4.57	25.3	-23	18.78	102.8	-50.91	21.41	109.3	-66	25.98	134.6	-89.21
	5	23	1.38	92.66	0.93	62.36	-0.24	2.882	2.96	32.3	-23	15.59	121.5	-40.73	16.92	132.4	-53	19.88	164.7	-75.98
	6	29	0	96.8	0	65.14	0	3.602	1.34	34.6	-23	-12.9	126.9	-30.55	-13.26	137.1	-40	-11.5	171.6	-62.75
	7	35	-1.4	92.66	-0.9	62.36	0.24	2.882	-3	32.3	-23	-16.1	121.5	-40.73	-17.67	132.4	-53	-19.1	164.7	-75.98
	8	41	-2.8	80.23	-1.9	53.99	0.48	0.722	-4.6	25.3	-23	-19.3	102.8	-50.91	-22.16	109.3	-66	-23.9	134.6	-89.21
	9	47	-4.1	59.51	-2.8	40.05	0.72	-2.88	-6.2	13.8	-23	-22.3	72.33	-61.09	-26.58	71.84	-79	-28.5	86.16	-102.4
	10	53	-5.5	30.5	-3.7	20.53	0.96	-7.92	-7.8	3.83	-29.2	-25	33	-71.27	-30.76	18.91	-93	-32.8	36.84	-121.8
	11	58	-6.7	6E-14	-4.5	0	1.2	-14.4	-9.4	7.67	-53.7	-27.3	20.34	-81.45	-33.96	26.44	-106	-36.7	34.11	-159.6
3	1	0	6.68	0	4.49	0	-1.44	-14.4	9.95	7.67	-53.7	27.45	20.34	-81.45	35.46	26.44	-106	45.41	34.11	-159.6
	2	5	5.53	30.5	3.72	20.53	-1.2	-6.48	8.33	6.9	-27.6	25.48	28.41	-57.2	32.39	23.8	-73	40.73	35.31	-101.1
	3	11	4.14	59.51	2.79	40.05	-0.96	0.001	6.72	18.4	-18.4	23.37	69.01	-50.84	28.86	66.77	-65	35.58	87.44	-83.69
	4	17	2.76	80.23	1.86	53.99	-0.72	5.041	5.11	32.3	-16.1	20.93	104.8	-44.49	24.92	113.6	-57	30.02	145.9	-73.23
	5	23	1.38	92.66	0.93	62.36	-0.48	8.641	3.49	41.5	-13.8	18.2	132	-38.13	20.64	147.8	-49	24.13	189.2	-62.77
	6	29	0	96.8	0	65.14	-0.24	10.8	1.88	46.1	-11.5	15.23	147.5	-31.78	16.08	164.7	-41	17.97	210.7	-52.31
	7	35	-1.4	92.66	-0.9	62.36	0	11.52	0.81	46.1	-9.21	-13.2	151.1	-25.42	-14.21	169.1	-33	-12.4	215.2	-41.85
	8	41	-2.8	80.23	-1.9	53.99	0.24	10.8	-2.4	41.5	-6.91	-16.4	140.8	-19.07	-18.4	157.7	-24	-18.8	199.2	-31.38
	9	47	-4.1	59.51	-2.8	40.05	0.48	8.64	-4	32.3	-4.61	-19.8	113	-12.71	-22.89	130.8	-16	-23.8	163	-20.92
	10	53	-5.5	30.5	-3.7	20.53	0.72	5.04	-5.6	18.4	-2.3	-23.2	66.38	-6.356	-27.6	78.86	-8.2	-28.9	97.29	-10.46
	11	58	-6.7	6E-14	-4.5	0	0.96	0	-7.3	0	0	-26.8	0	0	-32.48	0	0	-34	0	0



<u>Prestress Losses</u>:

$P_i = 0.7f_{pu}\Sigma A_{sp}$ $\qquad P_i = 404.838\ \text{kips}$

<u>Elastic Shortening:</u> Δf_{pES}

f_{cgp}= sum of concrete stresses at the center of gravity of prestressing tendons due to prestressing force at transfer and the self-weight of the member section at section of maximum moment, f_{cgp} will be calculated on M_g using the overall beam length at release

$$M_{max1} = SW_{girder}\frac{RS^2}{8} \qquad M_{max1} = 1.243\times 10^3\ \text{kips in}$$

$$f_{cgp1} = \left(\frac{P_i}{A}\right) + \left[\frac{P_i (e_{cc})^2}{I}\right] - \left(\frac{M_{max1} e_{cc}}{I}\right) \qquad f_{cgp1} = 1.705\,\text{ksi}$$

$$\Delta f_{pES} = \frac{E_p}{E_c(fc_{gr}, WI_{fgr})} f_{cgp1} \qquad \Delta f_{pES} = 11.336\,\text{ksi}$$

Note: in Conspan 2.1 output file they reported a wrong value for f_{cgp} but the right value for Δf_{cgp}

Shrinkage:

$$\Delta f_{pSR} = (17 - 0.15RH)\text{ksi} \qquad \Delta f_{pSR} = 5.75\,\text{ksi}$$

Creep: Δf_{pCR}

Δf_{cdp}= Change in concrete stress at center of gravity of prestressing due to pemenant loads except the loads acting at time of applying prestressing force calculated at the same section as f_{cgp}

$$\Delta f_{cdp} = \left[\left(MZ_{HD_5} \, 12 \frac{e_{cc}}{I}\right) + \left[MZ_{comp_5} \, 12 \frac{(y_{bc} - y_{bs})}{I_c}\right]\right] \qquad \Delta f_{cdp} = 0.255\,\text{ksi}$$

Now for the total final losses f_{cgp} will be conservatively computed on Mg using the design span length

$$M_{max2} = SW_{girder} \frac{BTB^2}{8} \qquad M_{max2} = 1.162 \times 10^3 \,\text{kips in}$$

$$f_{cgp2} = \left(\frac{P_i}{A}\right) + \left(\frac{P_i e_{cc}^{\ 2}}{I}\right) - \left(\frac{M_{max2} e_{cc}}{I}\right) \qquad f_{cgp2} = 1.728\,\text{ksi}$$

$$\Delta f_{pCR} = 12 f_{cgp2} - 7 \Delta f_{cdp} \qquad \Delta f_{pCR} = 18.949\,\text{ksi}$$

Relaxation of Prestressing Strands:

Relaxation at Transfer: Δf_{PR1}

$$\Delta f_{pR1} = \frac{\log\left(24\frac{RT}{day}\right)}{40}\left(\frac{f_j}{f_{py}} - 0.55\right)f_j \qquad \Delta f_{pR1} = 1.801\,\text{ksi}$$

Relaxation after Transfer:

$$\Delta f_{pR2} = 0.3\left[20 \quad - 0.4\Delta f_{pES} - 0.2\left(\Delta f_{pSR} + \Delta f_{pCR}\right)\right] \qquad \Delta f_{pR2} = 3.158\,\text{ksi}$$

Total Losses at Transfer:

$\Delta f_{pi} = \Delta f_{pES} + \Delta f_{pR1}$ $\qquad \Delta f_{pi} = 13.136\,\text{ksi}$

stress in tendons after transfer: $\quad f_{pt} = f_j - \Delta f_{pi} \qquad f_{pt} = 189.364\,\text{ksi}$

total prestressing losses after transfer: $\quad P_{pi} = f_{pt}\Sigma A_{sp} \qquad P_{pi} = 405.617\,\text{kips}$

initial loss% $\quad InLoss = \frac{\Delta f_{pi}}{f_j}100 \qquad InLoss = 6.487$ %

Total Losses at service load:

$$\Delta f_{pt} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2} \qquad \Delta f_{pt} = 39.192\,\text{ksi}$$

stress in tendons at service load: $\quad f_{pe} = f_j - \Delta f_{pt} \qquad f_{pe} = 163.308\,\text{ksi}$

total prestressing losses at service load: $\quad Ppe = f_{pe}\Sigma A_{sp} \qquad Ppe = 349.805\,\text{kips}$

total loss% $\quad TLoss = \frac{\Delta f_{pt}}{f_j}100 \qquad TLoss = 19.354$ %

Check of Concrete Stresses at Transfer:

$P_{pi} = 405.617\,\text{kips}$

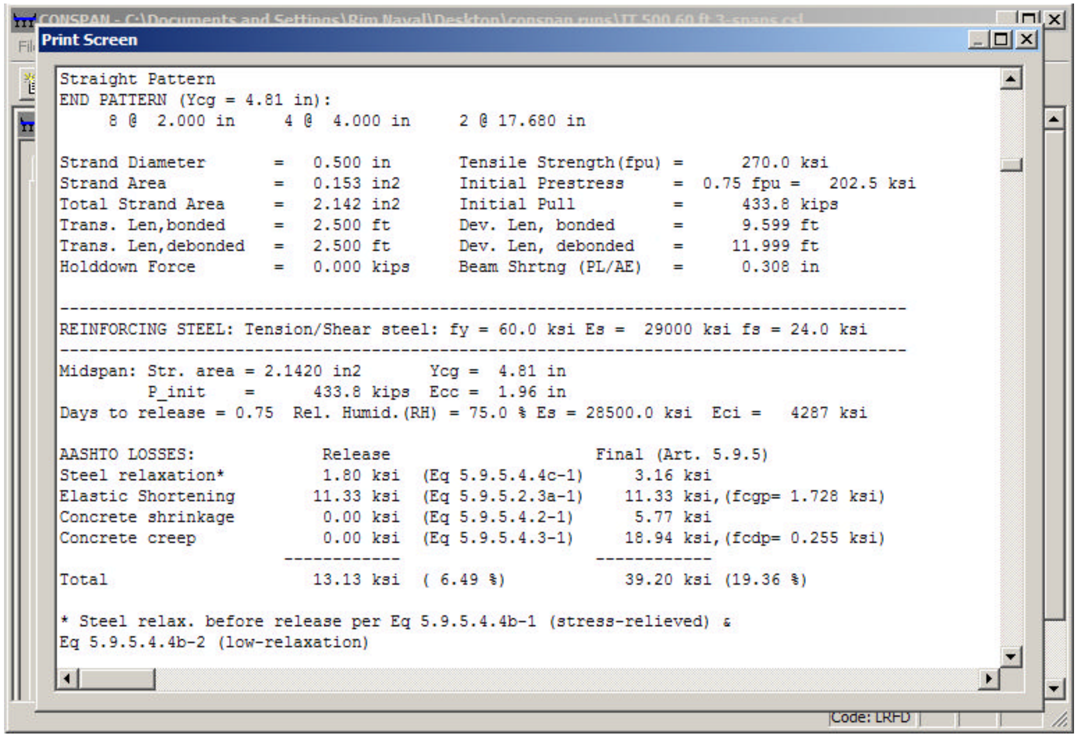
Stress Limits for Concrete:

Compression: $f_{cltrans.} = 0.6fc_{gr}$

$f_{cltrans.} = 3\,\text{ksi}$

Tension: $f_{tltrans.} = 0.0948\sqrt{fc_{gr}}$

$f_{tltrans.} = 0.212\,\text{ksi}$



Print Screen

```
Straight Pattern
END PATTERN (Ycg = 4.81 in):
     8 @  2.000 in     4 @  4.000 in     2 @ 17.680 in

Strand Diameter       =   0.500 in      Tensile Strength(fpu) =      270.0 ksi
Strand Area           =   0.153 in2     Initial Prestress     =  0.75 fpu =   202.5 ksi
Total Strand Area     =   2.142 in2     Initial Pull          =      433.8 kips
Trans. Len,bonded     =   2.500 ft      Dev. Len, bonded      =      9.599 ft
Trans. Len,debonded   =   2.500 ft      Dev. Len, debonded    =     11.999 ft
Holddown Force        =   0.000 kips    Beam Shrtng (PL/AE)   =      0.308 in

-------------------------------------------------------------------------------------
REINFORCING STEEL: Tension/Shear steel: fy = 60.0 ksi Es =  29000 ksi fs = 24.0 ksi
-------------------------------------------------------------------------------------
Midspan: Str. area = 2.1420 in2       Ycg =  4.81 in
         P_init    =      433.8 kips  Ecc =  1.96 in
Days to release = 0.75  Rel. Humid.(RH) = 75.0 % Es = 28500.0 ksi  Eci =   4287 ksi

AASHTO LOSSES:              Release                          Final (Art. 5.9.5)
Steel relaxation*            1.80 ksi  (Eq 5.9.5.4.4c-1)     3.16 ksi
Elastic Shortening          11.33 ksi  (Eq 5.9.5.2.3a-1)    11.33 ksi,(fcgp= 1.728 ksi)
Concrete shrinkage           0.00 ksi  (Eq 5.9.5.4.2-1)      5.77 ksi
Concrete creep               0.00 ksi  (Eq 5.9.5.4.3-1)     18.94 ksi,(fcdp= 0.255 ksi)
                        ------------                     ------------
Total                       13.13 ksi  ( 6.49 %)            39.20 ksi (19.36 %)

* Steel relax. before release per Eq 5.9.5.4.4b-1 (stress-relieved) &
Eq 5.9.5.4.4b-2 (low-relaxation)
```

Code: LRFD

Check of Stresses at Transfer Length Section:

Due to the camber of the beam at release, the beam self weight acts on the overall beam length

Transfer length from bearing

$\text{diff} = (PTP - BTB)0.5 \qquad \text{diff} = 1\,\text{ft}$

$TL = 60D - \text{diff} \qquad TL = 1.5\,\text{ft}$

$$M_{DLtrans.} = \left[0.5SW_{girder}RS(TL + \text{diff}) - SW_{girder}\left[(TL + \text{diff})^2\right]0.5\right]$$

$M_{DLtrans.} = 198.555\,\text{kips in}$

concrete stress at top fiber of the beam:

$$ft_{trans.} = \left(\frac{P_{pi}}{A} - P_{pi}\frac{e_{cc}}{s_t} + \frac{M_{DLtrans.}}{s_t} \right) \qquad ft_{trans.} = 0.729\,\text{ksi}$$

Check of Stresses at Mid-span Section:

$$M_{DLmspan.} = SW_{girder}\frac{RS^2}{8}12$$

$$M_{DLmspan.} = 1.492 \times 10^4\,\text{kips in}$$

$$ft_{mspan.} = \frac{P_{pi}}{A} - P_{pi}\frac{e_{cc}}{s_t} + \frac{M_{DLmspan.}}{12s_t} \qquad ft_{mspan.} = 2.668\,\text{ksi}$$

$$fb_{mspan.} = \frac{P_{pi}}{A} + P_{pi}\frac{e_{cc}}{s_b} - \frac{M_{DLmspan.}}{12s_b} \qquad fb_{mspan.} = 1.399\,\text{ksi}$$

Check of Concrete Stresses at Service Load:

$Ppe = 349.805\,\text{kips}$

Stress Limits for Concrete:

Compression:

Beam

Deck

$f_{clg.f.1} = 0.6fc_{gf}$	$f_{clg.f.1} = 4.8\,\text{ksi}$	$f_{cld.f.1} = 0.6fc_d$	$f_{cld.f.1} = 3\,\text{ksi}$
$f_{clg.f.2} = 0.45fc_{gf}$	$f_{clg.f.2} = 3.6\,\text{ksi}$	$f_{cld.f.2} = 0.45fc_d$	$f_{cld.f.2} = 2.25\,\text{ksi}$
$f_{clg.f.3} = 0.4fc_{gf}$	$f_{clg.f.3} = 3.2\,\text{ksi}$	$f_{cld.f.3} = 0.4fc_d$	$f_{cld.f.3} = 2\,\text{ksi}$

Tension:

$$f_{tg.s.} = \frac{6}{1000}\sqrt{fc_{gf}1000} \qquad f_{tg.s.} = 0.537\,\text{ksi} \qquad f_{td.s.} = \frac{6}{1000}\sqrt{fc_{d}1000}$$

Check of Stresses at Mid-span Section:

Service I

1. Final I : Under permenant and transit loads:

$$ft_{sI.mspan.} = \frac{Ppe}{A} - Ppe\frac{e_{cc}}{s_t} + \frac{MZ_{DL_5}12}{s_t} + \frac{MZ_{HD_5}12}{s_t} + \frac{MZ_{comp_5}12(y_{tc} - DT - HT)}{I_c} + \frac{PMZ_{LL_5}12(y_{tc} - DT - HT)}{I_c}$$

$$ft_{sI.mspan.} = 4.59\,\text{ksi}$$

2. Final II : Under permenant loads :

$$f_{td.s.} = 0.424\,\text{ksi}$$

$$ft_{sII.mspan.} = \frac{Ppe}{A} - Ppe\frac{e_{cc}}{s_t} + \frac{MZ_{DL_5}12}{s_t} + \frac{MZ_{HD_5}12}{s_t} + \frac{MZ_{comp_5}12(y_{tc} - DT - HT)}{I_c}$$

$$ft_{sII.mspan.} = 3.951\,\text{ksi}$$

3. Final III : Under Live loads plus one-half of Dead loads:

$$ft_{sIII.mspan.} = 0.5\left[\frac{Ppe}{A} - Ppe\frac{e_{cc}}{s_t} + \frac{MZ_{DL_5}12}{s_t} + \frac{MZ_{HD_5}12}{s_t} + \frac{MZ_{comp_5}12(y_{tc} - DT - HT)}{I_c}\right] + \frac{PMZ_{LL_5}12(y_{tc} - DT - HT)}{I_c}$$

$$ft_{sIII.mspan.} = 2.614\,\text{ksi}$$

Concrete stress at the top Fiber of the slab, Service I:

1. Final I : Under permenant and transit loads:

$$ft_{sI.smspan.} = \frac{MZ_{comp_5} 12n}{s_{tc}} + \frac{PMZ_{LL_5}(12)\,n}{s_{tc}}$$

$$ft_{sI.smspan.} = 1.031\,\text{ksi}$$

2. Final II : Under permenant loads:

$$ft_{sII.smspan.} = \frac{MZ_{comp_5} 12n}{s_{tc}}$$

$$ft_{sII.smspan.} = 0.05\,\text{ksi}$$

3. Final III : Under Live loads plus one-half of Dead loads:

$$ft_{sIII.smspan.} = \left[0.5\left(\frac{MZ_{comp_5} 12n}{s_{tc}}\right) + \frac{PMZ_{LL_5}(12)\,n}{s_{tc}}\right]$$

$$ft_{sIII.smspan.} = 1.006\,\text{ksi}$$

<u>Check of Tension Stress at the bottom fiber of the beam, Service III:</u>

$$fb_{sIII.mspan.} = \frac{Ppe}{A} + Ppe\frac{e_{cc}}{s_b} - \frac{MZ_{DL_5} 12}{s_b} - \frac{MZ_{HD_5} 12}{s_b} - \frac{MZ_{comp_5} 12}{s_{bc}} - \frac{\left(0.8PMZ_{LL}\right)_5 12}{s_{bc}}$$

$$fb_{sIII.mspan.} = -0.592\,\text{ksi}$$

<u>Check of Strength limit state:</u>

Positive Moment Section:

Total factored bending moment for strength I is:

$$PM_u = 1.25\left(MZ_{DL_5} + MZ_{HD_5} + MZ_{comp_5}\right) + 1.75\,PMZ_{LL_5}$$

$$PM_u = 584.716\,\text{kips in}$$

$d_p = h_c - y_{bs}$ $\quad$ $d_p = 21.619\,\text{in}$

assume the section behave as a rectangular section

$$c_{rec} = \frac{\Sigma A_{sp} f_{pu}}{0.85 fc_d \beta1(fc_d)\, bw + k\Sigma A_{sp} \frac{f_{pu}}{d_p}} \qquad c_{rec} = 6.492\,\text{in}$$

$$c_T = \frac{\Sigma A_{sp} f_{pu} - 0.85 fc_d \beta1(fc_d)(bw - ww)\, hf}{0.85 fc_d \beta1(fc_d)\, ww + k\Sigma A_{sp} \frac{f_{pu}}{d_p}} \qquad c_T = 7.515\,\text{in}$$

$$c = \text{if}(c_{rec} > hf, c_T, c_{rec}) \qquad c = 7.515\,\text{in}$$

Average stress in prestressing steel

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \qquad f_{ps} = 243.72\,\text{ksi}$$

$$a = \beta1(fc_d)\, c$$

$$M_n = \frac{\left[\Sigma A_{sp} f_{ps}\left(d_p - \frac{a}{2}\right) + 0.85 fc_d (bw - ww)\, \beta1(fc_d)\, hf\left(\frac{a}{2} - \frac{hf}{2}\right)\right]}{12}$$

$$M_n = 809.904\,\text{kips in}$$

Bridgewid	(ft)	:Overall bridge width
LCurb	(ft)	:Left curb width
RCurb	(ft)	:Right curb width
NL		:Number of lanes
LW	(ft)	:Lane width
ST	(in)	:Supplementry thickness
DT	(in)	:Deck thickness
HT	(in)	:Haunch thickness
HW	(in)	:Haunch width
PTP	(ft)	: Pier to pier length
PL	(ft)	:Precast length
BTB	(ft)	:Bearing to bearing length
PTPR	(ft)	: Pier to centerline length
RS	(ft)	:Release span
EC	(Ksi)	:Modulus of elasticity of the concrete
Wlfgr	(pcf)	:Unit length weight for the girder concrete at transfer
Wlfgf	(pcf)	:Unit length weight for the girder concrete after 28 days
Wlfd	(pcf)	:Unit length weight for the deck after 28 days
fcgr	(Ksi)	:Compression strength for the girder concrete at release
fcgf	(Ksi)	:Compression strength of the girder after 28 days
fcd	(Ksi)	:Compression strength of the deck after 28 days
υ		:Poisson's ratio
h	(in)	:Height of the non-composite section
bw	(in)	:Bottom flange width
ww	(in)	:Stem width
hf	(in)	:Flange height
A	(in^2)	:Area of the non-composite section
I	(in^4)	: Moment of inertia of the non-composite section
yb	(in)	: Distance between the c.g. of the non-composite section and the bottom of the section
yt	(in)	: Distance between the c.g. of the non-composite section and the top of the sectionhc

$$\mathrm{ksi} = 1000\frac{\mathrm{lb}}{\mathrm{in}^2}$$

$$\mathrm{kips} = \mathrm{lb}1000$$

$$\mathrm{ksf} = \frac{\mathrm{lb}1000}{\mathrm{ft}^2}$$

$$= \mathrm{kips}\ \mathrm{in}$$

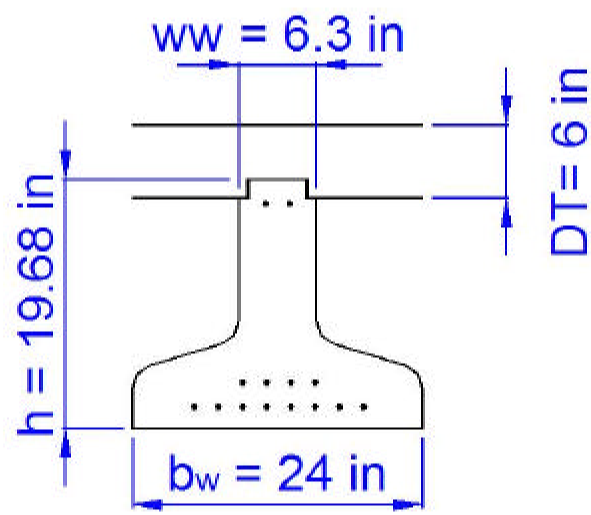
$$= \frac{\mathrm{in}}{\mathrm{kips}}$$

$$= \mathrm{ksi}$$

$$= \frac{\mathrm{ft}^{5.5}\mathrm{ksi}}{\mathrm{lb}^2}$$

hc	(in)	:Height of the composite section
Ac	(in^2)	:Area of the composite section
Ic	(in^4)	: Moment of inertia of the composite section
ybc	(in)	: Distance between the c.g. of the composite section and the bottom of the section
ytc	(in)	: Distance between the c.g. of the composite section and the top of the composite section

D	(in)	:Tendon Diameter
Asp	(in^2)	:Tendon area
Ep	(Ksi)	:Modulus of elastisity of the tendons
k		:Constant defined by AASHTO used in calculating the section's flexural capacity
fpu	(Ksi)	:Tendon's ultimate stress
fpy	(Ksi)	:Tendon's yielding stress
fj	(Ksi)	:jacking stress
ec1, ec2, ec2	(in)	:Eccentricity of the tendons rows
n1, n2, n3		:Number of tendons in a row
ybs	(in)	:Distance between the c.g. of the tendons and the bottom of the section.
ΣAsp	(in^2)	:Tendons area
RT	(day)	:Release time
RH	(%)	:Relative humidity
SW_{girder}	(kip/ft)	:Girder self weight
SW_{HD}	(Kip/ft)	:Haunch and deck self weight
ml	(Kip/ft)	:Maintainance load
PM_{DF}		:Positive moment distribution factor
NM_{DF}		:Negative moment distribution factor
V_{DF}		:Shear force distribution factor
IM		:Live load impact factor
pi	(Kips)	:Jacking force calculated based on 0.7fpu jacking stress used to calculate elastic shortening loss



f_{cgp1} (Ksi) :sum of concrete stresses at the center of gravity of prestressing tendons due to prestressing force at transfer and the self-weight of the member section at section of maximum moment

M_{max1} (Kips-in) :Bending moment calculated at the midspan of a simply supported girder due to self weight based on the overall beam length

Δf_{pES} (Ksi) :Elastic shortening loss

Δf_{pSR} (Ksi) :Shrinkage loss

Δf_{cdp} (Ksi) : Change in concrete stress at center of gravity of prestressing due to pemenant loads except the loads acting at time of applying prestressing force calculated at the same section as f_{cgp}

M_{max2} (Kips-in) :Bending moment calculated at the midspan of a simply supported girder due to self weight based on the design span length

Δf_{pcr} (Ksi) :Creep loss

Δf_{PR1} (Ksi) :Relaxation at transfer

Δf_{PR2} (Ksi) :Relaxation after transfer

Δf_{Pi} (Ksi) :Total losses at transfer

f_{pt} (Ksi) :Stress in tendons at transfer

p_{pi} (Ksi) :Prestress force in tendons at transfer

Δf_{Pt} (Ksi) :Total losses at service load

f_{pe} (Ksi) :Stress in tendons at service load

p_{pe} (Ksi) :Prestress force in tendons at service load

$f_{cltrans.}$ (Ksi) :Allowable compression stress at transfer

$f_{tltrans.}$ (Ksi) :Allowable tension stress at transfe

TL (ft) :Transfer length

$M_{DLtrans.}$ (Kip-in) :Dead load moment at transfer length

$ft_{trans.}$ (Ksi) :Stress at the top of the section at the transfer length section

$M_{DLmspan.}$ (Kip-in) :Dead load moment at midspan

$ft_{mspan.}$ (Ksi) :Stress at the top of the section at the midspan section

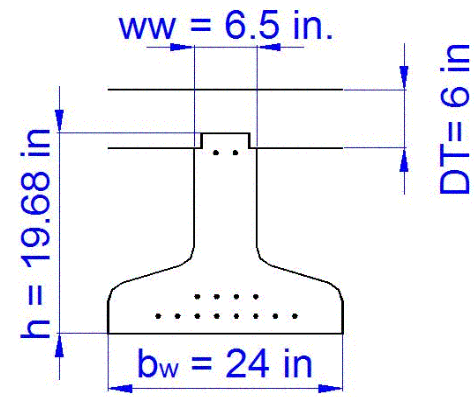
$fb_{mspan.}$ (Ksi) :Stress at the bottom of the section at the midspan section

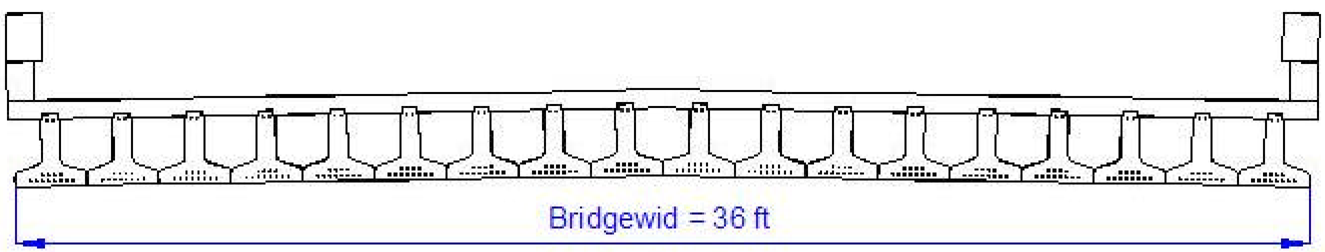
$f_{clg.f.2}$, $f_{clg.f.2}$, $f_{clg.f.2}$ (Ksi):Allowable compression stresses at service load final 1, 2, 3 respectively

$f_{tg.s.}$ (Ksi) :Allowable tension stresses at service load

$ft_{sI.mspan.}$, $ft_{sII.mspan.}$, $ft_{sIII.mspan.}$ (Ksi):Midspan stresses at the top of the section at service load final 1, 2, 3 respectively

$ft_{sI.smspan.}$, $ft_{sII.smspan.}$, $ft_{sIII.smspan.}$ (Ksi):Midspan stresses at the top of the slab at service load final 1, 2, 3 respectively





K - TRAN

KANSAS TRANSPORTATION RESEARCH
AND
NEW - DEVELOPMENTS PROGRAM

A COOPERATIVE TRANSPORTATION RESEARCH PROGRAM BETWEEN:

KANSAS DEPARTMENT OF TRANSPORTATION

THE UNIVERSITY OF KANSAS

KANSAS STATE UNIVERSITY